

AD-A101 711

HOWARD NEEDLES TAMMEN AND BERGENOFF NEW YORK F/G 13/75  
ENERGY DISSIPATOR FACILITIES AND RIPRAP REPAIR, COY GLEN AND CA--ETC(11)  
AUG 75 DACW49-75-C-0052

F/G 13/13

ENERGY DISSIPATOR FACILITIES AND RIPRAP REPAIR

AUG 75

COV GLEN AND CA=  
DACW48-75-C-0052

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20. ABSTRACT (Continue on reverse side if necessary and identify by block number)		
<p>The design analysis for this project provides detailed designs in four sections for the following items: (1) Two hydraulic drop structures and attached wingwall on Coy Glen; 2. Soils and foundation analysis for the above structures and cantiliver sheet pile wingwall alternates for the two drop structures; (3) Riprap repair for the section in Cayuga Channel between the Lehigh Valley Railroad bridge and the drop structure at Station 160+00; 4. Dynamic water loads on the drop structure and hydraulic design for Coy Glen by the Buffalo District. The design considered two types of wingwalls. The factor of</p>		

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✓ safety in bearing for the concrete wingwalls is not considered adequate and the more conservative steel sheet pile wingwalls are recommended.

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8

DESIGN ANALYSIS

ENERGY DISSIPATOR FACILITIES AND RIPRAP REPAIR

COY GLEN AND CAYUGA INLET

ITHACA, NEW YORK

CONTRACT NO. DACW49-75-C-0052

WORK ORDER NO. 1

DEPARTMENT OF THE ARMY

BUFFALO DISTRICT, CORPS OF ENGINEERS

AUGUST 1975

PREPARED BY

**HNTB**

HOWARD NEEDLES TAMMEN & BERGENDOFF

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## DESIGN ANALYSIS

### ENERGY DISSIPATOR FACILITIES AND RIPRAP REPAIR

COY GLEN AND CAYUGA INLET

ITHACA, NEW YORK

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## DESIGN ANALYSIS

### ENERGY DISSIPATOR FACILITIES AND RIPRAP REPAIR

COY GLEN AND CAYUGA INLET

ITHACA, NEW YORK

CONTRACT NO. DACW49-75-C-0052

WORK ORDER NO. 1

### SCOPE & GENERAL RECOMMENDATIONS

The design analysis for this project provides detailed designs in four sections for the following items.

1. Two hydraulic drop structures and attached wingwalls on Coy Glen.
2. Soils and foundation analysis for the above structures and cantiliver sheet pile wingwall alternates for the two drop structures.
3. Riprap repair for the section in Cayuga Channel between the Lehigh Valley Railroad bridge and the drop structure at Station 160+00.
4. Dynamic water loads on the drop structure and hydraulic design for Coy Glen by the Buffalo District.

The design considered two types of wingwalls. The factor of safety in bearing for the concrete wingwalls is not considered adequate and the more conservative steel sheet pile wingwalls are recommended.

#### 1. DROP STRUCTURE DESIGN

1.1 This design is for the two drop structures and concrete cantiliver wingwalls of Coy Glen.



1.2 The design of the drop structures was for two limiting loading conditions: (a) no flow (empty) with saturated soil; and (b) design flow with dynamic hydraulic impact. The hydraulic loads are based on a 50-year design flow.

1.3 The walls for the drop structure are designed for an at-rest earth pressure plus water pressure. Calculations for the wall design are on Sheets S-4 to 20.

1.4 At-rest lateral earth pressure plus water pressure was used for the design of the end sills. The calculations are on Sheets S-21 to 28.

1.5 The baffle floes in the bottom of the drop structures have been designed for a horizontal hydraulic dynamic force of 3,000 pounds each. The calculations are on Sheet S-29.

1.6 The bottom slab of the drop structures has been designed for normal dead load plus a vertical hydraulic dynamic load of 1,630 p.s.f. over a five-foot by 15-foot area. It was also checked against uplift from ground water pressure. The calculations are on Sheets S-30 to 42.

1.7 The wingwalls for the drop structure were designed for an active earth pressure plus water pressure. Three wall heights were designed, one for the downstream end of the structures and two for the upstream end of the structures. For the latter two wall designs one is founded at the same level as the drop structure and the other is founded five feet above this level. Calculations are on Sheets S-43 to 58.

1.8 Detailed plans, elevations, sections and construction procedures have been developed for the drop structure and the concrete cantilever wingwall alternates. These data are given on Sheets S-59 to 66.

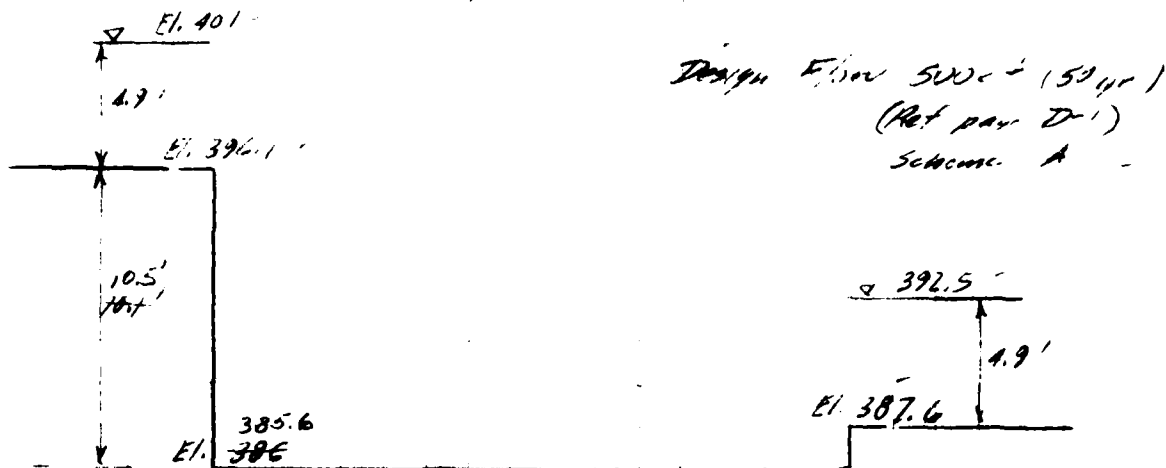
# HNTB

CALCULATIONS FOR

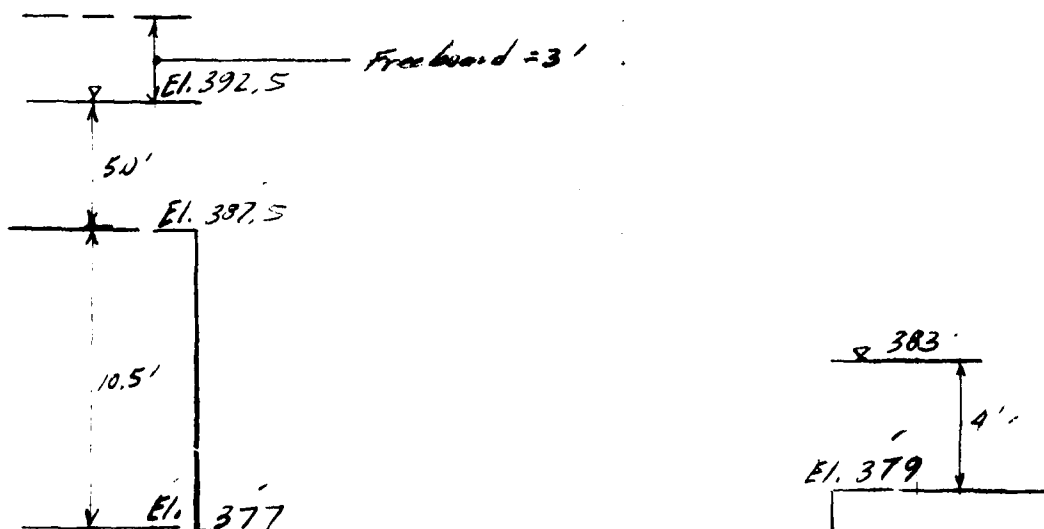
*Coy Glen, Ithaca, N.Y.*

MADE BY AAS-J DATE 4-2-75 JOB NO. 4204-29-01  
CHECKED BY C.D. DATE 4/24/75 SEC. NO. \_\_\_\_\_  
SHEET NO. 5-1

HOWARD NEEDLES TAMMEN & BERENSON  
CONSULTING ENGINEERS



Str. No. 1 (Incl. D-2)



Str. No. 2 (Incl. D-2)

- For Structure Geometry use Scheme 1A from Enclosure D-1 (Ref. 1.1, sh. D-1.)
- Allow 3' Freeboard (Encl. D-2)

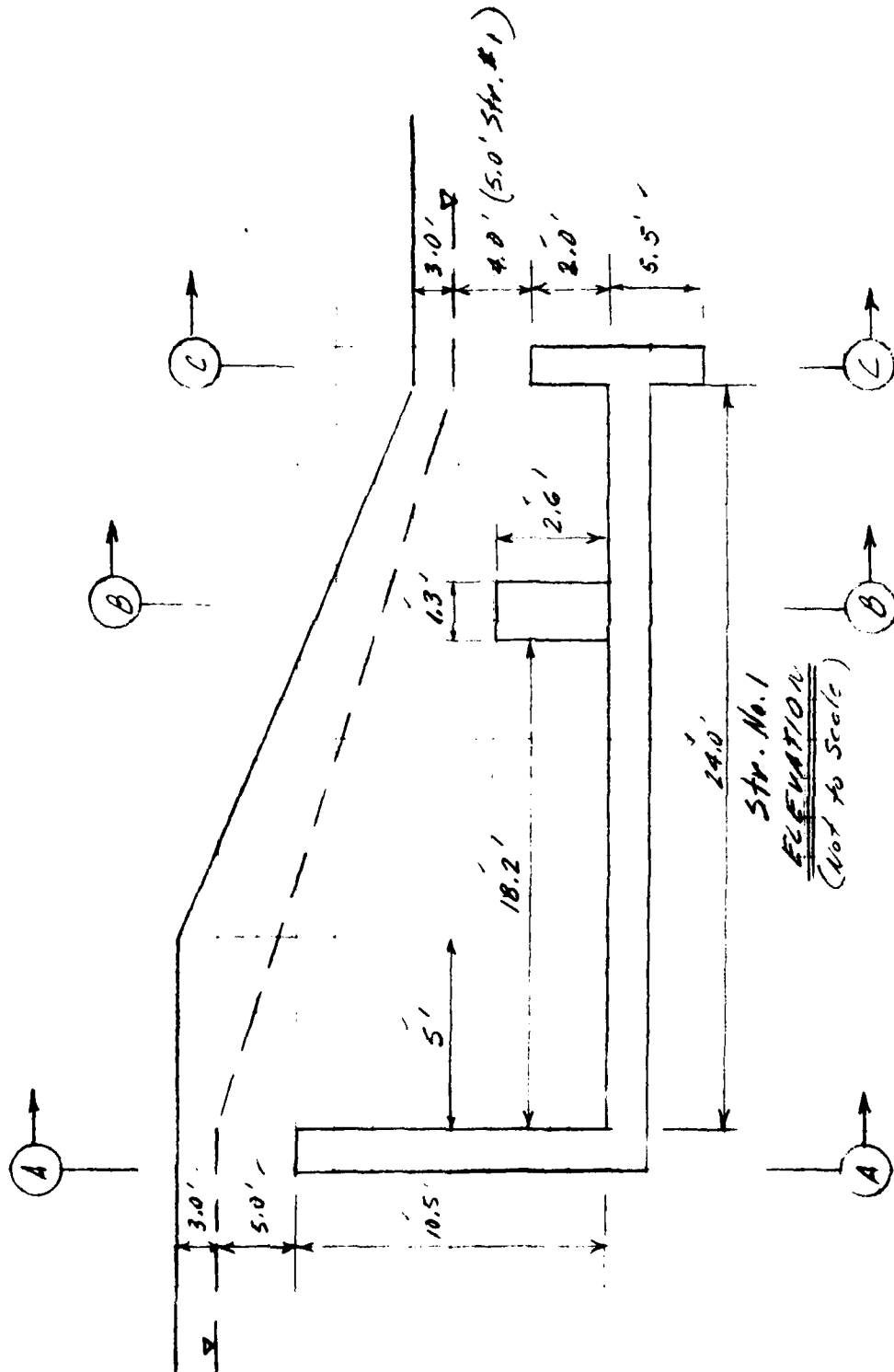
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CALCULATIONS FOR

*Coy Glen, Ithaca, N.Y.*

MADE BY MA5-J DATE 4-2-75 JOB NO. 4204  
CHECKED BY LD DATE 4/3/75 SEC. NO. \_\_\_\_\_  
SHEET NO. 5-2

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CONSULTING ENGINEERS



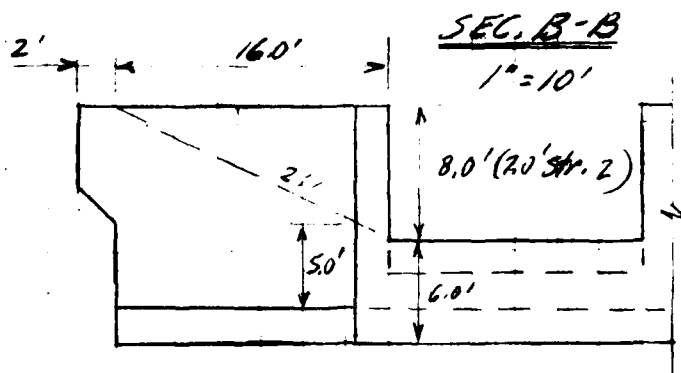
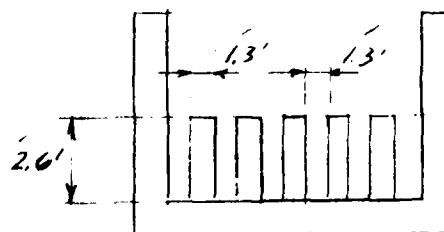
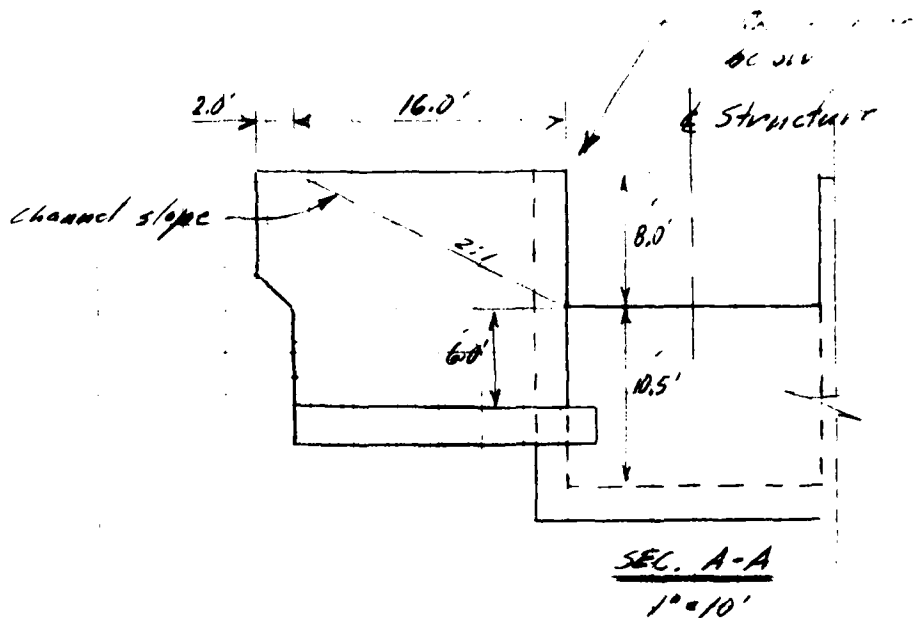
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CALCULATIONS FOR

Coy Glenn, Ithaca, N.Y.

MADE BY AAS-J DATE 4-2-75 JOB NO. 4204  
 CHECKED BY 12.1 DATE 1/6/75 SEC. NO.           
 SHEET NO. 5-3

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*Rounding at Inlet Wall*

$r = 0.6 d_c$  (per R. Gorecki - Buffalo Dist., 4-2-75)

$d_c = (2')^3$  per "Design of Small Dams" USBR, 2nd Ed., 1973

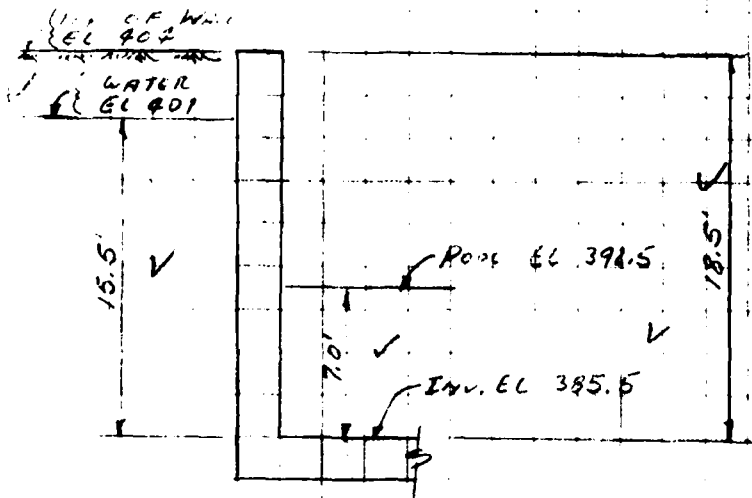
$d_c = \left[ \frac{(500/15)^2}{22.2} \right]^{1/3} = 3.26'$ ;  $r = 0.6 \times 3.26 = 1.96'$  USE 2.0'

# HNTB

CALCULATIONS FOR

MADE BY J. H. J. DATE 4/2/75 JOB NO. 9604  
CHECKED BY J. H. J. DATE 4/16/75 SEC. NO. 54  
SHEET NO. 54

STR No. 1 Box



FOR EL. SEE  
H, N, T & E. SM No. 1

HORIZ. C. & M. PRESS. = H.G.T.

SAT. UNIT PR.  $\sim p_s = 56.5 \text{ *}/10$  ✓

BOTTOM UNIT PR.  $\sim p_b = 35.3 \text{ *}/10$  ✓

UNIT WT. OF CONTENT  $\sim p_w = 62.8 \text{ *}/10$  ✓

H, N, T & E. SM No. 1

CASE a)  $\sim$  NO FLOW (EMPTY) WITH SAT. SOIL

CASE b)  $\sim$  DESIGN FLOW WITH DYNAMIC HYD. 34.

CASE a & b FROM RESUME OF NEGOTIATIONS.

CASE a & b ARE GROUP I COATING @ 100%  
REF. CITY 1110-1-1111 P. 1, 2 & PARA. 4 ✓

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# HNTB

CALCULATIONS FOR

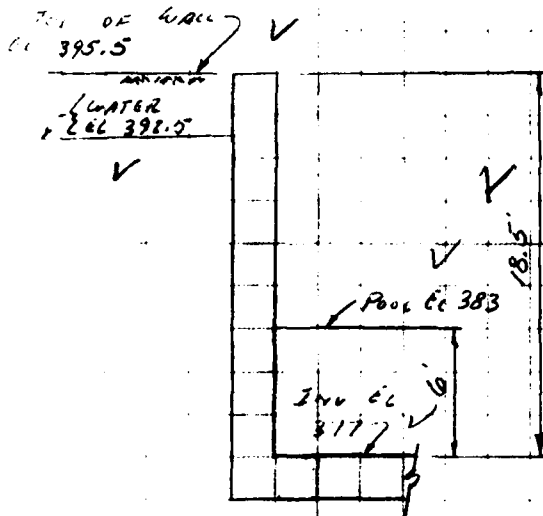
Cox Creek, Tammen, N.J.

MADE BY LD  
CHECKED BY JKT

DATE 4/16/75  
DATE 4/16/75

JOB NO. 100  
SEC. NO. 1  
SHEET NO. 5-5

STR NO 2 BOX



FOR EL 382  
H.N.T.S.B. 3/1/75

## CRITICAL DESIGN CASES & SOLUTIONS

CASE a) BOTH STR NO 1 & 2 ARE SIMILAR  
HENCE THE HEIGHTS ARE EQUAL  
H.C.P. ARE EQUAL

CASE b) SINCE THE HEIGHTS ARE EQUAL  
IS ALL STRUCTURES HAVE POOL DEPTH IS  
EQUAL IN STRUCTURE NO 2, IT IS  
OBTAINED THAT STRUCTURE NO 2 IS STRONGER  
FOR DESIGN. (6' POOL DEPTH)

CASE b') ALSO INVESTIGATE CASE b' WITH  
2' POOL DEPTH. THIS DESIGN IS  
FOR WALL DESIGN.

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# HNTB

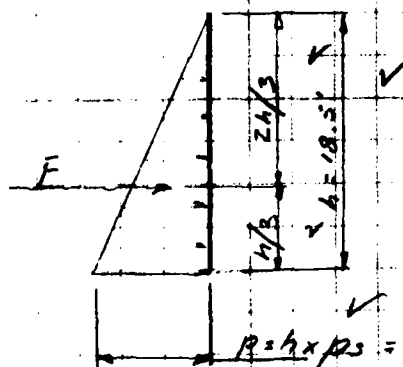
CALCULATIONS FOR

Cox Glen, Indiana, NY

MADE BY L.D. DATE 4/3/75 JOB NO. 4104  
 CHECKED BY Z.R.F. DATE 4/16/75 SEC. NO.         
 SHEET NO. 5-6

IND. CALC. 17/2/75

CASE a) STR. NO 1 & 2



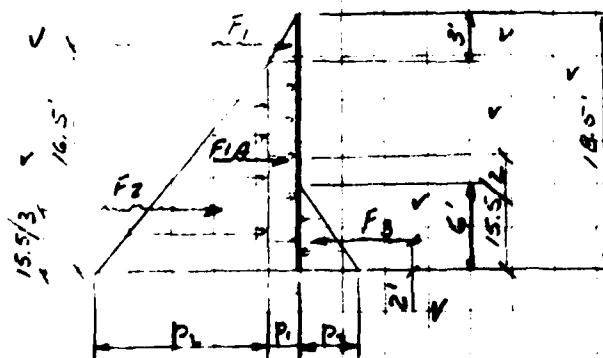
RESULT. FORCE = F ACTING  $h/3$  ABOVE BASE

$$F = P \times h \div 2 = 123.4 \times 18.5 \div 2 = 11380 \text{ lbs}$$

$$MOM = F \times h \div 3 = 11380 \times 18.5 \div 3 = 70,180 \text{ ft-lbs}$$

NOT Gov. MOM.

CASE b) STR. NO 2



$P_1$  = PRESS. DUE TO SAT. EARTH  
 $P_2$  = " " " WATER  
 BUOYANT EARTH  
 $P_3$  = PRESS. DUE TO FLOWING WATER IN BOX

HOWARD NEEDLES TAMMEN & BERGENDOFF CONSULTING ENGINEERS



# HNTB

CALCULATIONS FOR

Cox Glen, ITHACA, NY

MADE BY LD  
CHECKED BY JKT

DATE 4/15/75  
DATE 4/16/75

JOB NO. 1-4  
SEC. NO. 5-7  
SHEET NO. 5-7

$$p_1 = h \times p_s = 3 \times 66.5 = 200 \frac{\text{lb}}{\text{ft}}$$

$$p_2 = h \times (p_o + p_w) = 15.5 (35.3 + 62.4) = 1514 \frac{\text{lb}}{\text{ft}}$$

$$p_3 = h \times p_u = 6 \times 62.4 = 375 \frac{\text{lb}}{\text{ft}}$$

$$F_1 = 200 \times 3 \div 2 = 300 \checkmark$$

$$F_{1A} = 200 \times 15.5 = 3100 \checkmark$$

$$F_2 = 1514 \times 15.5 \div 2 = 11,735 \checkmark$$

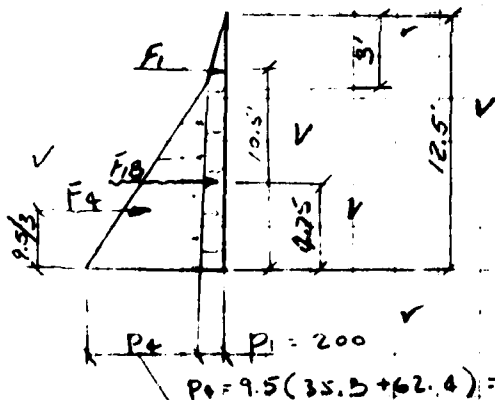
$$F_3 = 375 \times 6 \div 2 = 1,125 \checkmark$$

$$\Sigma F = 14,010 \checkmark$$

$$M_{\text{mom}} = 300 \times 16.5 + 3100 \times \frac{15.5}{2} + 11,735 \times \frac{15.5}{3} - 1,125 \times 2 =$$

$$= 87,355 \text{ lb-ft @ Top of FPA}$$

FIND MAX MOM 6' ABOVE INV.



$$F_1 = 200 \times 3 \div 2 = 300 \checkmark$$

$$F_{1A} = 200 \times 9.5 = 1900 \checkmark$$

$$F_2 = 928 \times 9.5 \div 2 = 4410 \checkmark$$

$$\Sigma F = 6,610 \checkmark$$

10

HOWARD NEEDLES TAMMEN & BERGENDOFF CONSULTING ENGINEERS

# HNTB

CALCULATIONS FOR

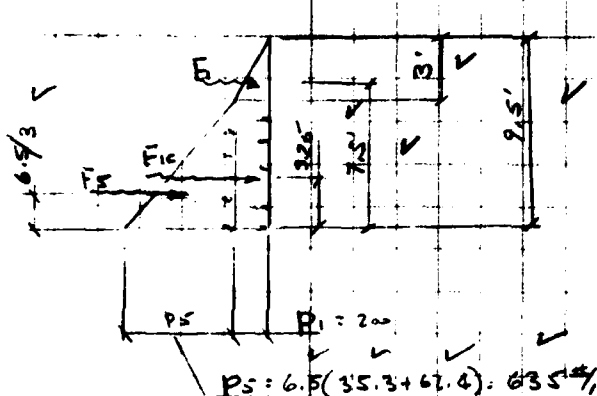
MADE BY C.D. DATE 7-11 JOB NO. 9204  
 CHECKED BY J.R. DATE 4-16-75 SEC. NO.   
 SHEET NO. 5-8

Cox Green, TACCA, NY

$$M_{max} = 300 \times 10.5 + 1900 \times 9.75 + 4410 \times 9.5 = 26,140 \text{ ft-lb}$$

6' ABOVE TOP OF PILE

FIND MAX MOM 9' ABOVE INV.



$$F_1 = 200 \times 3 \div 2 = 300$$

$$F_{10} = 200 \times 6.5 = 1300$$

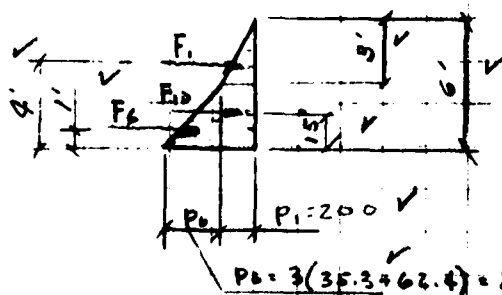
$$F_5 = 635 \times 6.5 \div 2 = 2065$$

$$\Sigma F = 3665$$

$$M_{om} = 300 \times 7.5 + 1300 \times 3.25 + 2065 \times \frac{6.5}{3} = 10,950 \text{ ft-lb}$$

9' ABOVE TOP OF PILE

FIND MAX MOM 12.5' ABOVE INV.



$$F_1 = 200 \times 3 \div 2 = 300$$

$$F_{10} = 200 \times 3 = 600$$

$$F_6 = 293 \times 3 \div 2 = 440$$

$$\Sigma F = 1340$$

HOWARD NEEDLES TAMMEN & BERGENDOFF CONSULTING ENGINEERS

# HNTB

CALCULATIONS FOR

MADE BY C.D. DATE 4/2/55 JOB NO. 4207  
CHECKED BY J.K.Z. DATE 4.16.75 SEC. NO.         
SHEET NO. 5-9

$$Mom = 500 \times 4 + 600 \times 1.5 + 440 \times 1 =$$

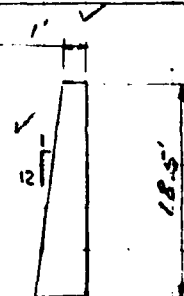
$$= 2540 \text{ ft-lb}$$

ABOVE GRADE,  $P_{H_2}$

Find Max Mom 15.5' ABOVE GRADE

$$Mom = F_1 \times 3 \div 3 = 300 \times 3 \div 3 = 300 \text{ ft-lb}$$

Find Grade Thickness @ Various Heights



@ TOP OF FTG  $t = 12'' + 18.5 \times 12 \div 12 = 30.5''$

@ 6' ABOVE TOP OF FTG  $t = 30.5 - 6 \times 12 \div 12 = 24.5''$

@ 9' do  $t = 30.5 - 9 \times 12 \div 12 = 21.5''$

@ 12.5' do  $t = 30.5 - 12.5 \times 12 \div 12 = 18.5''$

@ 15.5' do  $t = 30.5 - 15.5 \times 12 \div 12 = 15.0''$

@ 20' do  $t = 30.5 - 20 \times 12 \div 12 = 10.5''$

# HNTB

CALCULATIONS FOR

Cor GLENN ITHACA, N.Y.

MADE BY C.D.

DATE 4/26/75

JOB NO. 464

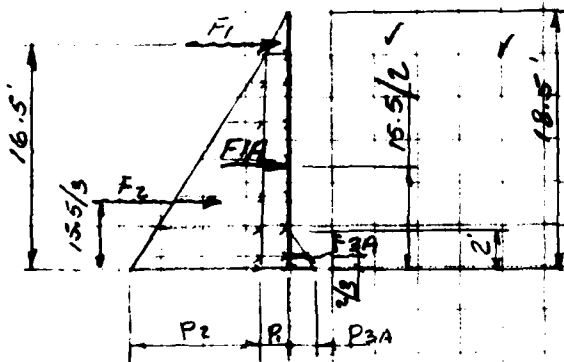
CHECKED BY J.K.

DATE 4/28/75

SEC. NO.

SHEET NO. 5-7H

CASE 5) ✓ STR NO 2 ✓



P<sub>1</sub> = PRESS. DUE TO SAT. EARTH ✓  
P<sub>2</sub> = " " " WATER & " ✓  
BUOYANT. EARTH ✓  
P<sub>3A</sub> = PRESS. DUE TO C' ✓  
POOL OF WATER IN ✓  
BOX ✓

$$\left. \begin{aligned} P_1 &= 200 \#/\text{ft} \checkmark \\ P_2 &= 1514 \#/\text{ft} \checkmark \\ P_{3A} &= 2 \times 62.4 = 125 \#/\text{ft} \checkmark \end{aligned} \right\} \text{SEE STR NO 7 } \checkmark$$

$$\left. \begin{aligned} F_1 &= 300 \# \checkmark \\ F_{1A} &= 3,100 \# \checkmark \\ F_L &= 11,250 \# \checkmark \end{aligned} \right\} \text{SEE STR NO 7 } \checkmark$$

$$F_{3A} = -125 \checkmark = (125 \times 2 \div 2) \checkmark$$

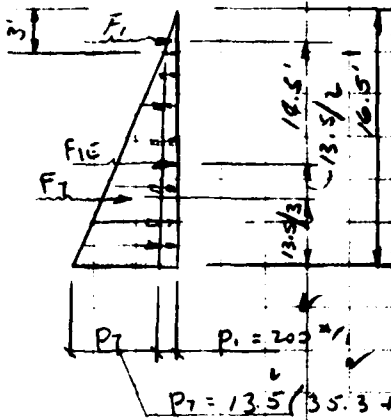
$$\Sigma F = 15,100 \# \checkmark$$

$$\begin{aligned} \text{MOM} &= 300 \times 16.5 + 3,100 \times \frac{15.5}{2} + 11,250 \times \frac{15.5}{3} - \\ &\quad - 125 \times \frac{2}{3} = 89,520 \text{ ft-lb } \checkmark \end{aligned}$$

HOWARD NEEDLES TAMMEN & BERGENDOFF CONSULTING ENGINEERS

CALCULATIONS FOR Box 51671 ITHACA NY

F 1111	M. 4	2	13	6	21
--------	------	---	----	---	----



$F_1 = 3.00$

$$F_{1c} = 2700 = 13.5 \times 200$$

$$F_2 = 810 \text{ kg} = 1319 \times 13.5 \div 2$$

$$\Sigma F = 1190 \text{ N} \checkmark$$

$$140m = 300 \times 14.5 + 2700 \times \frac{13.5}{6} + 5205 \times \frac{12.5}{3}$$

62,650 ✓ 2

About 9', 12.5' & 15.5' above ground level  
 are the three main levels of the site  
 on the site.

# HNTB

CALCULATIONS FOR

MADE BY 1.1 DATE 4/1/75 JOB NO. 0204  
 CHECKED BY J.R.S. DATE 4/16/75 SEC. NO. 5-10  
 SHEET NO. 5-10

Box Girder 1 TWIN

$f'_c = 3000 \text{ psi}$  (Pg 3, # 6 C.M. 1110-1-2101)  
 $f_c = .35 f'_c = 1050 \text{ psi}$  (Pg 4 # 7a d.p.)  
 GRADE 40 STEEL  $f_y = 40,000 \text{ psi}$  (Pg 4 # 4 C.M. 1110-2-2103)  
 $f_c = 20,000 \text{ psi}$  (Pg 4 # 7a C.M. 1110-1-2101)  
 WALL  $C_u C_v = 9"$  (Pg 2 # 3 C.M. 1110-2-2101)  
 $j = .891$  (Rein. Conc. Design Handbook) **ACI SP-3**  
 TABLE 1

Assume  $C_u$  For #10 Bar

$$4" + 1.27/2 = 4.64"$$

$$\therefore d = C - 4.64"$$

- $d_{4\#10} = 30.5 - 4.64 = 25.86"$
- $d_{3\#10} = 28.5 - 4.64 = 23.86"$
- $d_{2\#10} = 24.5 - 4.64 = 19.86"$
- $d_{1\#10} = 21.5 - 4.64 = 16.86"$
- $d_{11\#5} = 18.0 - 4.64 = 13.36"$
- $d_{15\#5} = 15.0 - 4.64 = 10.36"$

HOWARD NEEDLES TAMMEN & BERGENDOFF CONSULTING ENGINEERS

# HNTB

CALCULATIONS FOR

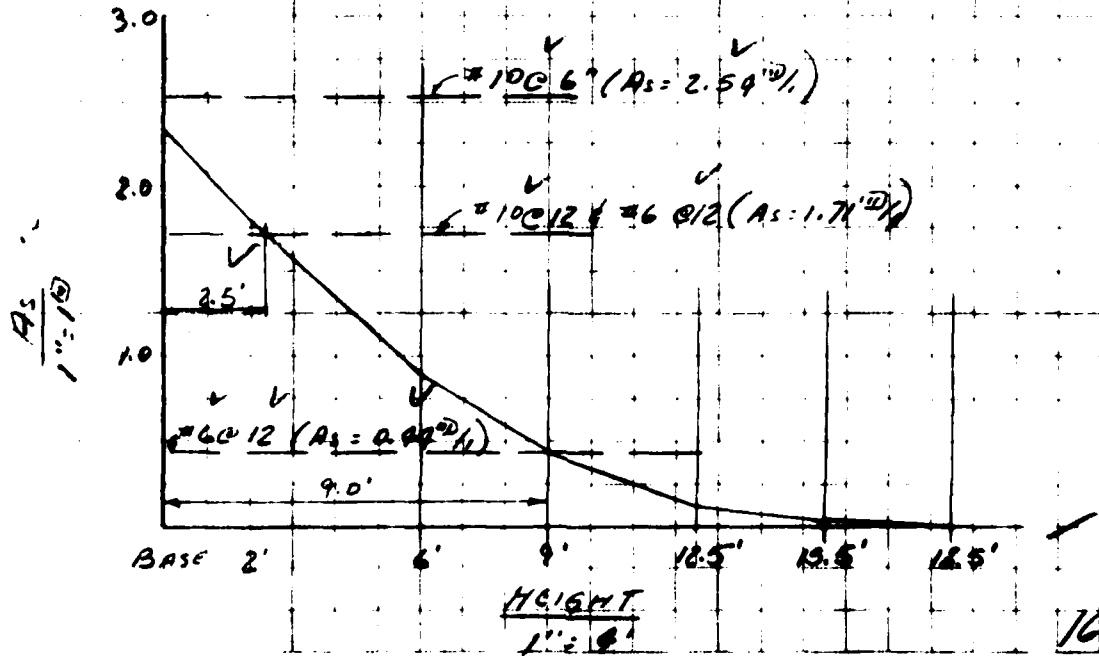
Coy Green, Indiana, 117

MADE BY L.D. DATE 7/2/75 JOB NO. 4209  
 CHECKED BY J.K.T. DATE 7/6/75 SEC. NO.         
 SHEET NO. 5-11

$$A_s = M / f_s \times j \times d$$

$A_s$  = AREA OF REIN. STEEL REQ'D. ( $\text{in}^2/\text{ft}$ )  
 $M$  = DESIGN MOM. ( $\text{in-k}/\text{ft}$ )  
 $f_s$  = ALLOW. STRESS IN REIN. STEEL ( $\text{k}/\text{in}^2$ )  
 $j$  = RATIO OF DIST. ( $j \cdot d$ ) BETWEEN RESULTANTS OF COMPRESSIVE & TENSILE STRESSES TO EFFECTIVE DEPTH.

$d$ : EFFECTIVE DEPTH (IN)	✓
$A_s$ BASE	$89.52 \times 12 \div 20 \times 891 \times 25.86 = 2.33 \text{ in}^2/\text{ft}$ ✓
$A_s$ 2'	$62.05 \times 12 \div \quad \times 23.86 = 1.77 \text{ in}^2/\text{ft}$ ✓
$A_s$ 6'	$26.10 \times 12 \div \quad \times 19.86 = 0.89 \text{ in}^2/\text{ft}$ ✓
$A_s$ 9'	$10.95 \times 12 \div \quad \times 16.36 = 0.44 \text{ in}^2/\text{ft}$ ✓
$A_s$ 12.5'	$2.58 \times 12 \div \quad \times 13.36 = 0.13 \text{ in}^2/\text{ft}$ ✓
$A_s$ 15.5'	$0.30 \times 12 \div \quad \times 10.36 = 0.02 \text{ in}^2/\text{ft}$ ✓



HOWARD NEEDLES TAMMEN & BERENDSON CONSULTING ENGINEERS

# HNTB

CALCULATIONS FOR

Cox Green TIRACH, N.Y.

MADE BY J.K.J.  
CHECKED BY J.K.J.

DATE 4/7/75  
DATE 4.17.25

JOB NO. 4204

SEC. NO.

SHEET NO. 5-12

DEV. LENGTH OF DEFORMED BARS (ACI STD 318-71)  
(#12.15 #43)

$$0.04 \times A_s f_y / (f_c')^2$$

#5	~	0.04 x .31 x 40,000 / (3000) <sup>2</sup>	= 9.1"	✓
#6	~	.04	= 12.9"	✓ G.O.
#7	~	.60	= 17.5"	✓ G.O.
#8	~	.72	= 23.1"	✓ G.O.
#9	~	1.0	= 29.2"	✓ G.O.
#10	~	1.27	= 37.1"	✓ G.O.
#11	~	1.56	= 45.6"	✓ G.O.

$$0.0004 \times d \times f_y$$

#5	~	0.0004 x .625 x 40,000	= 10.0"	✓ G.O.
#6	~	.75	= 12.0"	✓
#7	~	.875	= 14.0"	✓
#8	~	1.000	= 16.0"	✓
#9	~	1.128	= 18.0"	✓
#10	~	1.270	= 20.3"	✓
#11	~	1.410	= 22.6"	✓

\* ACI STD 318-71 Pg 44 #12.5 (d) ✓  
\*\* " " " Pg 18 #7.6 ✓

SPACING LENGTH OF #10 = .8 x 1.3 x 37.1 = 38.6" (USE 39")

" " " #6 = 1.3 x 12 = 15.6" (USE 16")

\*\*\* SINCE .8 x 12.5 < 12" USE 12" (ACI Pg 44 & 11.4)

LENGTH OF #10 DOUBLE ENDING  $E_{10} = (0.5' = 20') +$   
 $+ 39" = 69" (USE 6'-0")$

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# HNTB

CALCULATIONS FOR

CY GREEN, ITHACA, N.Y.

MADE BY 1.11 DATE 7.17.73 JOB NO. 4604  
CHECKED BY J.R.J. DATE 7.17.73 SEC. NO. 5-13  
SHEET NO. 5-13

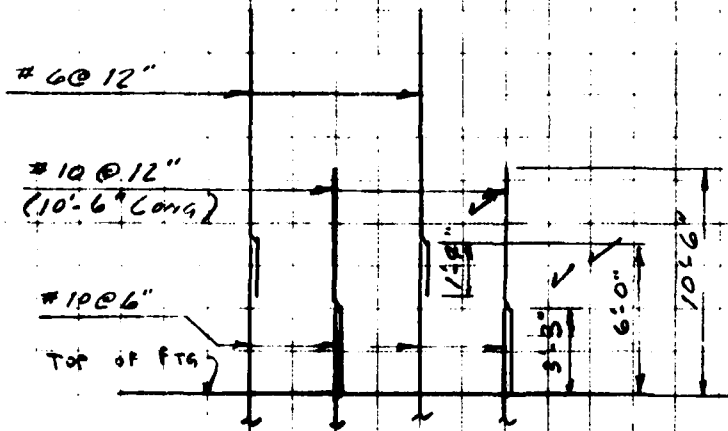
DETERMINE WHERE ONLY #6 ARE REQUIRED.

HOT SID 313-71  $R_d$  42 # 12.14

dc 9' = 16.86" (see 511 11) + G.V.

12# = 12 x 1.27 = 15.24

END # 10 = 9.0 + 16.86/12 = 10.41' ABOVE INV.



CONSULTING ENGINEERS

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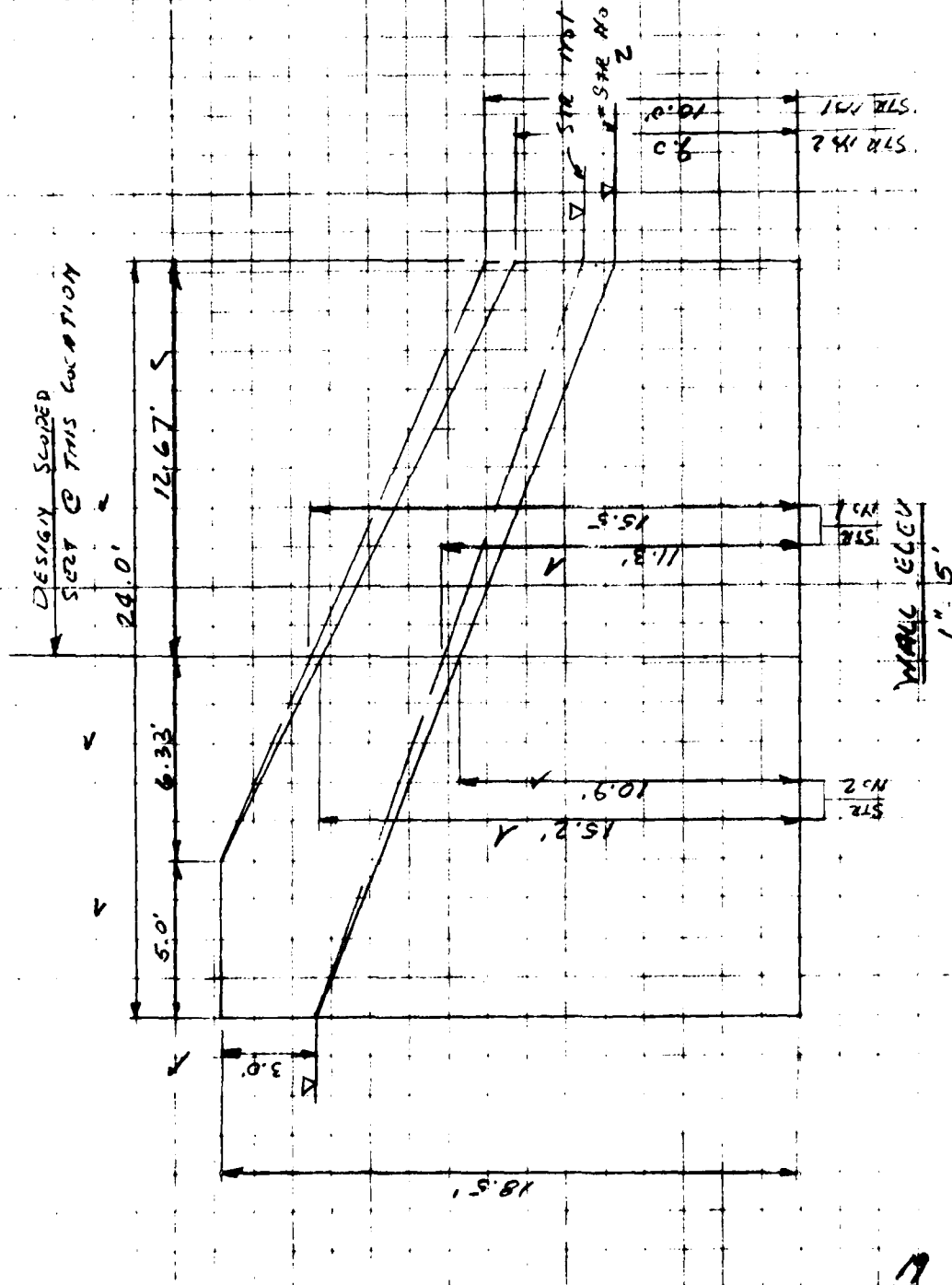
# HNTB

CALCULATIONS FOR

601 GLEN, TITACIA, N.Y.

MADE BY C.D. DATE 4/7/75 JOB NO. 4204  
 CHECKED BY 727 DATE 4/7/75 SEC. NO. 5-14  
 SHEET NO. 5-14

DESIGN SLOPED SECTION OF 12' WALL C 2/3 OF  
 ITS TOTAL HEIGHT FROM ELEV. ON THIS SHEET. IT  
 CAN BE SEEN THAT STR NO 2 SPORES DESIGN  
 SINCE WATER HEIGHT IN POOL IS 6' DEEP.



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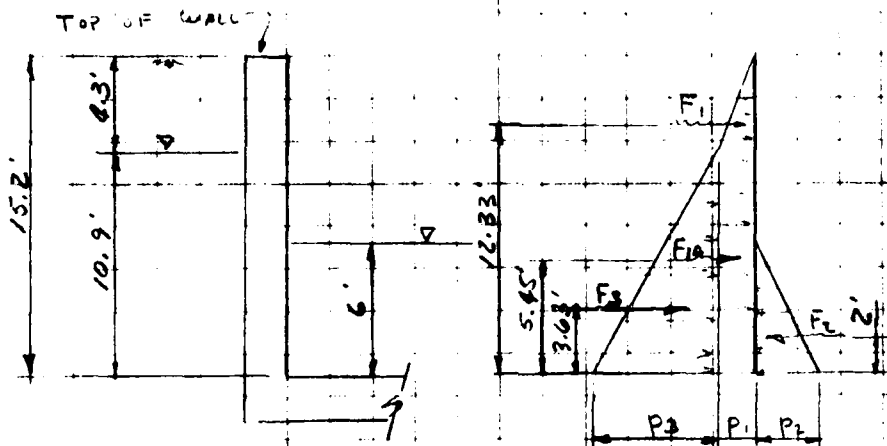
# HNTB

CALCULATIONS FOR

MADE BY L.V. DATE 8/11/75 JOB NO. 7604  
CHECKED BY J.K.J. DATE 4/17/78 SEC. NO.         
SHEET NO. 5-15

COY GREEN 17HCHN, 1

CASE 6) STR. 113 E



$$P_1 = h \times p_s = 4.3 \times 66.5 = 285 \text{ #/ft} \quad \checkmark$$

$$P_2 = h \times p_w = 6 \times 62.4 = 375 \text{ #/ft} \quad \checkmark$$

$$P_3 = h \times (p_s + p_w) = 10.9 \times (35.3 + 31.4) = 1065 \text{ #/ft} \quad \checkmark$$

$$F_1 = 285 \times 4.3 \div 2 = 615 \text{ #} \quad \checkmark$$

$$F_{1A} = 285 \times 10.9 = 3119 \text{ #} \quad \checkmark$$

$$F_2 = 375 \times 6 \div 2 = 1125 \text{ #} \quad \checkmark$$

$$F_3 = 1065 \times 12.33 \div 2 = 6595 \text{ #} \quad \checkmark$$

$$\Sigma F = 8405 \text{ #} \quad \checkmark$$

$$M_{\text{wall}} = 615 \times 7.33 + 3119 \times 1.45 + 1125 \times 3.63 + 6595 \times 3.63 = 1125 \text{ #} \times 43.355 \text{ #} \quad \checkmark$$

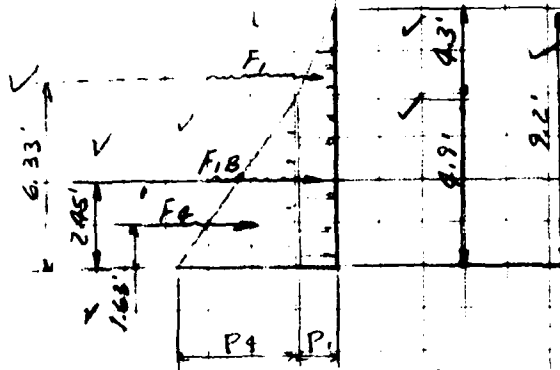
# HNTB

CALCULATIONS FOR

MADE BY C.D. DATE 4/1/75 JOB NO. 404  
 CHECKED BY J.K.T. DATE 4.17.75 SEC. NO. 5-16  
 SHEET NO. 5-16

Box Section, 21" x 21" x 1/2"

FIELD WIDTH 6' ABOVE INCL.



$$p_1 = h \times p_s = 4.3 \times 66.5 = 285 \text{ psi}$$

$$p_2 = h \times (p_s + p_w) = 4.9 (35.3 + 68.1) = 480 \text{ psi}$$

$$F_1 = 285 \times 4.3 \div 2 = 615$$

$$F_2 = 285 \times 4.9 = 1395$$

$$F_3 = 480 \times 4.9 \div 2 = 1175$$

$$\Sigma F = 3185$$

$$MOMENT = 615 \times 6.33 + 1395 \times 2.95 + 1175 \times 1.63 = 9225$$

SINCE AS READ WITH THE MAIN USE ONLY TWO POINTS TO POINT AS/AIRIGHT DIA. 1/2". (THEY ARE NOT CORRELATIVE)

# HNTB

CALCULATIONS FOR

MADE BY C.D. DATE 9/26/75 JOB NO. 9004  
 CHECKED BY J.A.F. DATE 4.28.75 SEC. NO.   
 SHEET NO. 5-16A

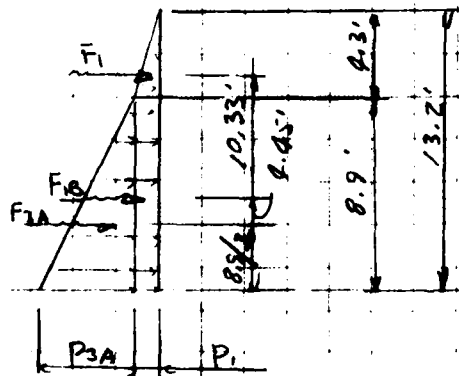
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CASE 6) STR 11.2 ✓

ADJUST WITH 6' POOL DEPTH  
 Pool DEPTH

$$\begin{aligned} \text{SH NO 15 } 6' \text{ POOL } 2' \text{ POOL } \checkmark \\ \text{P1200 BASE} = 43,355 \text{ lb} + 1125 \times 2' - 2 \times 62.4 \times \frac{1}{3} = \\ = 45,520 \text{ lb} \checkmark \end{aligned}$$

Pool 2' POOL 11.2



$$\begin{aligned} P1 &= 285 \text{ lb} \checkmark \text{ See SH 15} \\ P3A &= 8.9(35.3 + 62.4) = 870 \text{ lb} \checkmark \\ F1 &= 615 \text{ lb} \checkmark \text{ See SH 15} \\ F1R &= 295 \times 8.9 = 2,535 \text{ lb} \checkmark \\ F3A &= 870 \times 8.9 \div 2 = 3870 \text{ lb} \checkmark \end{aligned}$$

$$\begin{aligned} \text{11.2' } 2' \text{ CP} &= 615 \times 13.33 + 2,535 \times 0.95 + \\ &+ 3870 \times \frac{8.9}{3} = 29,115 \text{ lb} \checkmark \end{aligned}$$

# HNTB

CALCULATIONS FOR

COY GREEN, INDIAN, T.

MADE BY

DATE

JOB NO.

CHECKED BY

DATE

SEC. NO.

SHEET NO.

5-12

Assume  $\phi_c$  FOR #8 BAR ✓

$$9" \times 1.0/2 = 4.5" \checkmark$$

$$d = c - 4.5" \checkmark$$



$$d_{BASE} = 12 \times (15.2 \times 12 \div 12) - 4.5 = 22.7" \checkmark$$

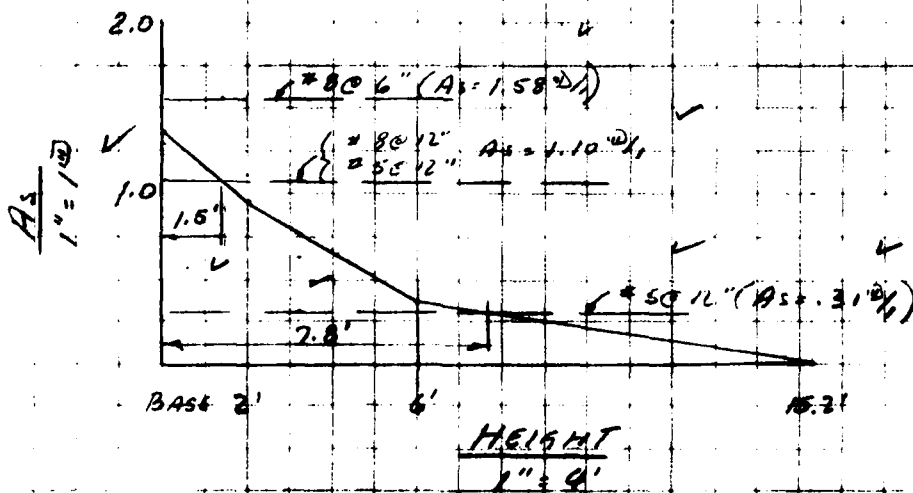
$$d_{TOP} = 22.7 - (8 \times 12 \div 12) = 20.7" \checkmark$$

$$d_{G'} = 22.7 - (6 \times 12 \div 12) = 16.7" \checkmark$$

$$A_s_{BASE} = 45.52 \times 12 \div 20 \times .891 \times 22.7 = 1.35 \text{ in}^2 \checkmark$$

$$A_s_{TOP} = 29.12 \times 12 \div 20 \times .891 \times 20.7 = 0.95 \text{ in}^2 \checkmark$$

$$A_s_{G'} = 9.23 \times 12 \div 20 \times .891 \times 16.7 = 0.37 \text{ in}^2 \checkmark$$



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# HNTB

MADE BY C.D. DATE 1/10/75 JOB NO. 9204  
 CHECKED BY J.R.T. DATE 4.17.75 SEC. NO.         
 SHEET NO. 5-18

CALCULATIONS FOR

Cox Hill, 2THACH, N.I.

HNT&B SH NO 12 AND ACI 318-71 ELC-7-G + 12-T

SPICE LENGTH OF #8 =  $.8 \times 23.1 \times 1.3 = 24"$  (USE 2'-0")

∴ MAKE SHORT DOUG 2'-0" ALSO  $F_{TG} > P_d = 23.1"$

SPICE LENGTH OF #5 =  $7.8 \times 12 = 15.6"$  SAY 1'-4"

∴ MAKE LONG DOUG 2'-0" + 1'-4" = 3'-4" ALSO  $F_{TG}$

1-2-3-4-5-6-7-8-9-10-11-12-13-14-15-16-17-18-19-20-21-22-23-24-25-26-27-28-29-30-31-32-33-34-35-36-37-38-39-40-41-42-43-44-45-46-47-48-49-50-51-52-53-54-55-56-57-58-59-60-61-62-63-64-65-66-67-68-69-70-71-72-73-74-75-76-77-78-79-80-81-82-83-84-85-86-87-88-89-90-91-92-93-94-95-96-97-98-99-100-101-102-103-104-105-106-107-108-109-110-111-112-113-114-115-116-117-118-119-120-121-122-123-124-125-126-127-128-129-130-131-132-133-134-135-136-137-138-139-140-141-142-143-144-145-146-147-148-149-150-151-152-153-154-155-156-157-158-159-160-161-162-163-164-165-166-167-168-169-170-171-172-173-174-175-176-177-178-179-180-181-182-183-184-185-186-187-188-189-190-191-192-193-194-195-196-197-198-199-200-201-202-203-204-205-206-207-208-209-210-211-212-213-214-215-216-217-218-219-220-221-222-223-224-225-226-227-228-229-230-231-232-233-234-235-236-237-238-239-240-241-242-243-244-245-246-247-248-249-250-251-252-253-254-255-256-257-258-259-260-261-262-263-264-265-266-267-268-269-270-271-272-273-274-275-276-277-278-279-280-281-282-283-284-285-286-287-288-289-290-291-292-293-294-295-296-297-298-299-300-301-302-303-304-305-306-307-308-309-310-311-312-313-314-315-316-317-318-319-320-321-322-323-324-325-326-327-328-329-330-331-332-333-334-335-336-337-338-339-340-341-342-343-344-345-346-347-348-349-350-351-352-353-354-355-356-357-358-359-360-361-362-363-364-365-366-367-368-369-370-371-372-373-374-375-376-377-378-379-380-381-382-383-384-385-386-387-388-389-390-391-392-393-394-395-396-397-398-399-400-401-402-403-404-405-406-407-408-409-410-411-412-413-414-415-416-417-418-419-420-421-422-423-424-425-426-427-428-429-430-431-432-433-434-435-436-437-438-439-440-441-442-443-444-445-446-447-448-449-450-451-452-453-454-455-456-457-458-459-460-461-462-463-464-465-466-467-468-469-470-471-472-473-474-475-476-477-478-479-480-481-482-483-484-485-486-487-488-489-490-491-492-493-494-495-496-497-498-499-500-501-502-503-504-505-506-507-508-509-510-511-512-513-514-515-516-517-518-519-520-521-522-523-524-525-526-527-528-529-530-531-532-533-534-535-536-537-538-539-540-541-542-543-544-545-546-547-548-549-550-551-552-553-554-555-556-557-558-559-560-561-562-563-564-565-566-567-568-569-570-571-572-573-574-575-576-577-578-579-580-581-582-583-584-585-586-587-588-589-590-591-592-593-594-595-596-597-598-599-600-601-602-603-604-605-606-607-608-609-610-611-612-613-614-615-616-617-618-619-620-621-622-623-624-625-626-627-628-629-630-631-632-633-634-635-636-637-638-639-640-641-642-643-644-645-646-647-648-649-650-651-652-653-654-655-656-657-658-659-660-661-662-663-664-665-666-667-668-669-670-671-672-673-674-675-676-677-678-679-680-681-682-683-684-685-686-687-688-689-690-691-692-693-694-695-696-697-698-699-700-701-702-703-704-705-706-707-708-709-710-711-712-713-714-715-716-717-718-719-720-721-722-723-724-725-726-727-728-729-730-731-732-733-734-735-736-737-738-739-740-741-742-743-744-745-746-747-748-749-750-751-752-753-754-755-756-757-758-759-760-761-762-763-764-765-766-767-768-769-770-771-772-773-774-775-776-777-778-779-780-781-782-783-784-785-786-787-788-789-790-791-792-793-794-795-796-797-798-799-800-801-802-803-804-805-806-807-808-809-810-811-812-813-814-815-816-817-818-819-820-821-822-823-824-825-826-827-828-829-830-831-832-833-834-835-836-837-838-839-840-841-842-843-844-845-846-847-848-849-850-851-852-853-854-855-856-857-858-859-860-861-862-863-864-865-866-867-868-869-870-871-872-873-874-875-876-877-878-879-880-881-882-883-884-885-886-887-888-889-890-891-892-893-894-895-896-897-898-899-900-901-902-903-904-905-906-907-908-909-910-911-912-913-914-915-916-917-918-919-920-921-922-923-924-925-926-927-928-929-930-931-932-933-934-935-936-937-938-939-940-941-942-943-944-945-946-947-948-949-950-951-952-953-954-955-956-957-958-959-960-961-962-963-964-965-966-967-968-969-970-971-972-973-974-975-976-977-978-979-980-981-982-983-984-985-986-987-988-989-990-991-992-993-994-995-996-997-998-999-1000-1001-1002-1003-1004-1005-1006-1007-1008-1009-1010-1011-1012-1013-1014-1015-1016-1017-1018-1019-1020-1021-1022-1023-1024-1025-1026-1027-1028-1029-1030-1031-1032-1033-1034-1035-1036-1037-1038-1039-1040-1041-1042-1043-1044-1045-1046-1047-1048-1049-1050-1051-1052-1053-1054-1055-1056-1057-1058-1059-1060-1061-1062-1063-1064-1065-1066-1067-1068-1069-1070-1071-1072-1073-1074-1075-1076-1077-1078-1079-1080-1081-1082-1083-1084-1085-1086-1087-1088-1089-1090-1091-1092-1093-1094-1095-1096-1097-1098-1099-1100-1101-1102-1103-1104-1105-1106-1107-1108-1109-1110-1111-1112-1113-1114-1115-1116-1117-1118-1119-1120-1121-1122-1123-1124-1125-1126-1127-1128-1129-1130-1131-1132-1133-1134-1135-1136-1137-1138-1139-1140-1141-1142-1143-1144-1145-1146-1147-1148-1149-1150-1151-1152-1153-1154-1155-1156-1157-1158-1159-1160-1161-1162-1163-1164-1165-1166-1167-1168-1169-1170-1171-1172-1173-1174-1175-1176-1177-1178-1179-1180-1181-1182-1183-1184-1185-1186-1187-1188-1189-1190-1191-1192-1193-1194-1195-1196-1197-1198-1199-1200-1201-1202-1203-1204-1205-1206-1207-1208-1209-1210-1211-1212-1213-1214-1215-1216-1217-1218-1219-1220-1221-1222-1223-1224-1225-1226-1227-1228-1229-1230-1231-1232-1233-1234-1235-1236-1237-1238-1239-1240-1241-1242-1243-1244-1245-1246-1247-1248-1249-1250-1251-1252-1253-1254-1255-1256-1257-1258-1259-1260-1261-1262-1263-1264-1265-1266-1267-1268-1269-1270-1271-1272-1273-1274-1275-1276-1277-1278-1279-1280-1281-1282-1283-1284-1285-1286-1287-1288-1289-1290-1291-1292-1293-1294-1295-1296-1297-1298-1299-1300-1301-1302-1303-1304-1305-1306-1307-1308-1309-1310-1311-1312-1313-1314-1315-1316-1317-1318-1319-1320-1321-1322-1323-1324-1325-1326-1327-1328-1329-1330-1331-1332-1333-1334-1335-1336-1337-1338-1339-1340-1341-1342-1343-1344-1345-1346-1347-1348-1349-1350-1351-1352-1353-1354-1355-1356-1357-1358-1359-1360-1361-1362-1363-1364-1365-1366-1367-1368-1369-1370-1371-1372-1373-1374-1375-1376-1377-1378-1379-1380-1381-1382-1383-1384-1385-1386-1387-1388-1389-1390-1391-1392-1393-1394-1395-1396-1397-1398-1399-1400-1401-1402-1403-1404-1405-1406-1407-1408-1409-1410-1411-1412-1413-1414-1415-1416-1417-1418-1419-1420-1421-1422-1423-1424-1425-1426-1427-1428-1429-1430-1431-1432-1433-1434-1435-1436-1437-1438-1439-1440-1441-1442-1443-1444-1445-1446-1447-1448-1449-1450-1451-1452-1453-1454-1455-1456-1457-1458-1459-1460-1461-1462-1463-1464-1465-1466-1467-1468-1469-1470-1471-1472-1473-1474-1475-1476-1477-1478-1479-1480-1481-1482-1483-1484-1485-1486-1487-1488-1489-1490-1491-1492-1493-1494-1495-1496-1497-1498-1499-1500-1501-1502-1503-1504-1505-1506-1507-1508-1509-1510-1511-1512-1513-1514-1515-1516-1517-1518-1519-1520-1521-1522-1523-1524-1525-1526-1527-1528-1529-1530-1531-1532-1533-1534-1535-1536-1537-1538-1539-1540-1541-1542-1543-1544-1545-1546-1547-1548-1549-1550-1551-1552-1553-1554-1555-1556-1557-1558-1559-1560-1561-1562-1563-1564-1565-1566-1567-1568-1569-1570-1571-1572-1573-1574-1575-1576-1577-1578-1579-1580-1581-1582-1583-1584-1585-1586-1587-1588-1589-1590-1591-1592-1593-1594-1595-1596-1597-1598-1599-1600-1601-1602-1603-1604-1605-1606-1607-1608-1609-1610-1611-1612-1613-1614-1615-1616-1617-1618-1619-1620-1621-1622-1623-1624-1625-1626-1627-1628-1629-1630-1631-1632-1633-1634-1635-1636-1637-1638-1639-1640-1641-1642-1643-1644-1645-1646-1647-1648-1649-1650-1651-1652-1653-1654-1655-1656-1657-1658-1659-1660-1661-1662-1663-1664-1665-1666-1667-1668-1669-1670-1671-1672-1673-1674-1675-1676-1677-1678-1679-1680-1681-1682-1683-1684-1685-1686-1687-1688-1689-1690-1691-1692-1693-1694-1695-1696-1697-1698-1699-1700-1701-1702-1703-1704-1705-1706-1707-1708-1709-1710-1711-1712-1713-1714-1715-1716-1717-1718-1719-1720-1721-1722-1723-1724-1725-1726-1727-1728-1729-1730-1731-1732-1733-1734-1735-1736-1737-1738-1739-1740-1741-1742-1743-1744-1745-1746-1747-1748-1749-1750-1751-1752-1753-1754-1755-1756-1757-1758-1759-1760-1761-1762-1763-1764-1765-1766-1767-1768-1769-1770-1771-1772-1773-1774-1775-1776-1777-1778-1779-1780-1781-1782-1783-1784-1785-1786-1787-1788-1789-1790-1791-1792-1793-1794-1795-1796-1797-1798-1799-1800-1801-1802-1803-1804-1805-1806-1807-1808-1809-1810-1811-1812-1813-1814-1815-1816-1817-1818-1819-1820-1821-1822-1823-1824-1825-1826-1827-1828-1829-1830-1831-1832-1833-1834-1835-1836-1837-1838-1839-1840-1841-1842-1843-1844-1845-1846-1847-1848-1849-1850-1851-1852-1853-1854-1855-1856-1857-1858-1859-1860-1861-1862-1863-1864-1865-1866-1867-1868-1869-1870-1871-1872-1873-1874-1875-1876-1877-1878-1879-1880-1881-1882-1883-1884-1885-1886-1887-1888-1889-1890-1891-1892-1893-1894-1895-1896-1897-1898-1899-1900-1901-1902-1903-1904-1905-1906-1907-1908-1909-1910-1911-1912-1913-1914-1915-1916-1917-1918-1919-1920-1921-1922-1923-1924-1925-1926-1927-1928-1929-1930-1931-1932-1933-1934-1935-1936-1937-1938-1939-1940-1941-1942-1943-1944-1945-1946-1947-1948-1949-1950-1951-1952-1953-1954-1955-1956-1957-1958-1959-1960-1961-1962-1963-1964-1965-1966-1967-1968-1969-1970-1971-1972-1973-1974-1975-1976-1977-1978-1979-1980-1981-1982-1983-1984-1985-1986-1987-1988-1989-1990-1991-1992-1993-1994-1995-1996-1997-1998-1999-2000-2001-2002-2003-2004-2005-2006-2007-2008-2009-2010-2011-2012-2013-2014-2015-2016-2017-2018-2019-2020-2021-2022-2023-2024-2025-2026-2027-2028-2029-2030-2031-2032-2033-2034-2035-2036-2037-2038-2039-2040-2041-2042-2043-2044-2045-2046-2047-2048-2049-2050-2051-2052-2053-2054-2055-2056-2057-2058-2059-2060-2061-2062-2063-2064-2065-2066-2067-2068-2069-2070-2071-2072-2073-2074-2075-2076-2077-2078-2079-2080-2081-2082-2083-2084-2085-2086-2087-2088-2089-2090-2091-2092-2093-2094-2095-2096-2097-2098-2099-2100-2101-2102-2103-2104-2105-2106-2107-2108-2109-2110-2111-2112-2113-2114-2115-2116-2117-2118-2119-2120-2121-2122-2123-2124-2125-2126-2127-2128-2129-2130-2131-2132-2133-2134-2135-2136-2137-2138-2139-2140-2141-2142-2143-2144-2145-2146-2147-2148-2149-2150-2151-2152-2153-2154-2155-2156-2157-2158-2159-2160-2161-2162-2163-2164-2165-2166-2167-2168-2169-2170-2171-2172-2173-2174-2175-2176-2177-2178-2179-2180-2181-2182-2183-2184-2185-2186-2187-2188-2189-2190-2191-2192-2193-2194-2195-2196-2197-2198-2199-2200-2201-2202-2203-2204-2205-2206-2207-2208-2209-2210-2211-2212-2213-2214-2215-2216-2217-2218-2219-2220-2221-2222-2223-2224-2225-2226-2227-2228-2229-2230-2231-2232-2233-2234-2235-2236-2237-2238-2239-2240-2241-2242-2243-2244-2245-2246-2247-2248-2249-2250-2251-2252-2253-2254-2255-2256-2257-2258-2259-2260-2261-2262-2263-2264-2265-2266-2267-2268-2269-2270-2271-2272-2273-2274-2275-2276-2277-2278-2279-2280-2281-2282-2283-2284-2285-2286-2287-2288-2289-2290-2291-2292-2293-2294-2295-2296-2297-2298-2299-2300-2301-2302-2303-2304-2305-2306-2307-2308-2309-2310-2311-2312-2313-2314-2315-2316-2317-2318-2319-2320-2321-2322-2323-2324-2325-2326-2327-2328-2329-2330-2331-2332-2333-2334-2335-2336-2337-2338-2339-2340-2341-2342-2343-2344-2345-2346-2347-2348-2349-2350-2351-2352-2353-2354-2355-2356-2357-2358-2359-2360-2361-2362-2363-2364-2365-2366-2367-2368-2369-2370-2371-2372-2373-2374-2375-2376-2377-2378-2379-2380-2381-2382-2383-2384-2385-2386-2387-2388-2389-2390-2391-2392-2393-2394-2395-2396-2397-2398-2399-2400-2401-2402-2403-2404-2405-2406-2407-2408-2409-2410-2411-2412-2413-2414-2415-2416-2417-2418-2419-2420-2421-2422-2423-2424-2425-2426-2427-2428-2429-2430-2431-2432-2433-2434-2435-2436-2437-2438-2439-2440-2441-2442-2443-2444-2445-2446-2447-2448-2449-2450-2451-2452-2453-2454-2455-2456-2457-2458-2459-2460-2461-2462-2463-2464-2465-2466-2467-2468-2469-2470-2471-2472-2473-2474-2475-2476-2477-2478-2479-2480-2481-2482-2483-2484-2485-2486-2487-2488-2489-2490-2491-2492-2493-2494-2495-2496-2497-2498-2499-2500-2501-2502-2503-2504-2505-2506-2507-2508-2509-2510-2511-2512-2513-2514-2515-2516-2517-2518-2519-2520-2521-2522-2523-2524-2525-2526-2527-2528-2529-2530-2531-2532-2533-2534-2535-2536-2537-2538-2539-2540-2541-2542-2543-2544-2545-2546-2547-2548-2549-2550-2551-2552-2553-2554-2555-2556-2557-2558-2559-2560-2561-2562-2563-2564-2565-2566-

# HNTB

CALCULATIONS FOR

JOY GUEN, ITHACA, N.Y.

MADE BY LD DATE 4/8/75 JOB NO. 4204  
CHECKED BY JKF DATE 4/18/75 SEC. NO. 5-19  
SHEET NO. 5-19

SHEAR IN WALL ✓

REF - ACI BUILDING CODE (318-71) ✓  
Pg 25 # 8.10.3 ✓

ALLOW SHEAR @ 55% AS GIVEN IN SPEC ✓

Pg 37 # 11.4.1 ✓

$$V_c = .55 \times 2 \times (f'_c) \sqrt{c} = .55 \times 2 \times (3000) \sqrt{c} = 60 \text{ psi}$$

TO COMPARE  $V_m$  (ACTUAL STRESS) IN WALL  
SEE Pg. 21 # 11.16.1 ✓

$$V_m = V_u \div \phi \text{ h d } \checkmark$$

ACI Pg 24 ✓  
# 8.10  $\phi = 1.2$

$V_u$  = DESIGNED SHEAR ✓  
 $\phi = 1$  ✓  
 $d = .8 P_u = .8 \times 16 = 12.8$  (APR 71) ✓  
 $h$  = THICKNESS OF SECTION ✓

$$\therefore V_m = V_u \div 1 \times h \times 9.6 = .104 V_u \div h$$

SET @ 72" ON 175 mm. DIAGONALLY BRUCKEN

FOR 18.5' SECT.  $V_m = .104 \times 14,010 \div 30.5 = 47.8 < 60$  ✓  
SEE SH 2 ✓

FOR SECT. SECT.  $V_m = .104 \times 9470 \div 27.2 = 36.2 < 60$  ✓  
SEE SH 15 ✓

HOWARD NEEDLES TAMMEN & BERGENDOFF CONSULTING ENGINEERS



# HNTB

CALCULATIONS FOR

COY GILCO, ITHACA, N.Y.

MADE BY L.D. DATE 4/9/75 JOB NO. 4204  
 CHECKED BY JRT DATE 4/18/75 SEC. NO. 5-20  
 SHEET NO. 5-20

TEMP + SHRINKAGE REINF IN BOX WALLS

EM 1110-2-2103 Pg 4 PARA 10

FOR HORIZ. STEEL USE 106(3) LOWER PORTION  
 106(1) UPPER PORTION

RESTRAINED EDGE  $\approx 26'$  LONG

$$\therefore 26 \div 4 = 6.5' \text{ OF REINFT}$$

CROSS SECT AREA @ HIGH END OF WALL =

$$(12 + 30.5) \times 18.5 \times \frac{1}{2} = 418''^2$$

$$\text{REQD AREA}/F_T = \frac{418''^2 \times 0.004}{2 \times 18.5} = 0.51''^2/\text{ft}$$

$$\text{USE } 2 @ 12'' \quad (A_s = .60''^2/\text{ft})$$

$$\text{MIN. REINFT PORTION} \quad A_s = .51''^2/\text{ft} \div 2 = .26''^2/\text{ft}$$

$$\text{USE } 2 @ 12'' \quad (A_s = .31''^2/\text{ft})$$

FOR VERT. STEEL USE 106(1) (IN FACE)

USE AREA OF SECT.  $\frac{2}{3}$  DOWN FOR MIN. HEIGHT

$$\therefore \text{THICKNESS} = 12'' + \left(\frac{2}{3} \times 18.5 \times 12 \div 2\right) = 24.3''$$

$$\text{STEEL AREA}/F_T = 12'' \times 24.3'' \times 0.002 \div 2 = .29''^2/\text{ft}$$

SINCE THE WALL WILL BE SUBJECTED TO  
 IMPACT AND TURBULENT FLOW USE #3 @ 12''  
 WHICH IS MAX  $(A_s = .40''^2/\text{ft})$

# HNTB

CALCULATIONS FOR

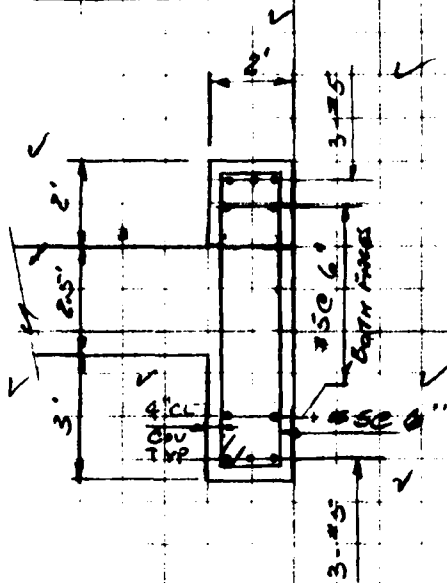
COY GLEN, ITHACA, NY

MADE BY L.P. DATE 9/1/75 JOB NO. 4202  
 CHECKED BY J.R.T. DATE 4.18.75 SEC. NO. 5-21  
 SHEET NO. 5-21

CONSULTING ENGINEERS

HOWARD NEEDLES TAMMEN & BERGENDOFF

END SILL (DOWNSIDE)



REF: - C.F.I. 1110 - 2-2403

ASSUME 10' HATCH FROM  
 GARTH C.V. DIMENSIONED  
 SILL

SAY  $d = 7'$   $L = 15'$

FOR BOT. REINF. ASSUME END  
 SILL IS SIMPLY SUPPORTED.

FOR TOP REINF. ASSUME END  
 SILL IS FIXED. E. C.I. 1115

EOT

$$\begin{aligned} \text{WATER} &= 2 \times 5' \times 62.4 = 624 \text{ lbs} \\ \text{BEAM} &= 2 \times 7.5' \times 150 = 2250 \text{ lbs} \\ &2874 \text{ lbs} = W \end{aligned}$$

$$M = 2874 \times 15^2 \div 8 = 80,830 \text{ lb-in} = 80.83 \text{ k}$$

$$V = 2874 \times 15 \div 2 = 21,555 \text{ lbs} = 21.6 \text{ k}$$

$$A_s = 80.83 \div 20 \times .391 \times 7 = .65 \text{ in}^2 \text{ USE } 3 \text{ \#5 } A_s = .93 \text{ in}^2$$

TOP

SINGLE THE DESIGN FROM W.C. BE SILL  
 (M =  $W L^2 \div 12$ ) USE 3-#5 BARS IN TOP OF  
 END SILL

# HNTB

CALCULATIONS FOR

COY REEF, THATCHER, N.Y.

MADE BY C.T. DATE 1/14/77 JOB NO. 4204  
 CHECKED BY J.H.T. DATE 4-18-75 SEC. NO.         
 SHEET NO. 5-22

CHECK REBAR ✓  
 REF ACT 318-71 ✓

\*  $P_b$  39 PARA 11.9 ✓  
 \*\*  $P_b$  25 PARA 8.10.3 ✓  
 \*\*\*  $P_b$  37 PARA 11.9.1 ✓

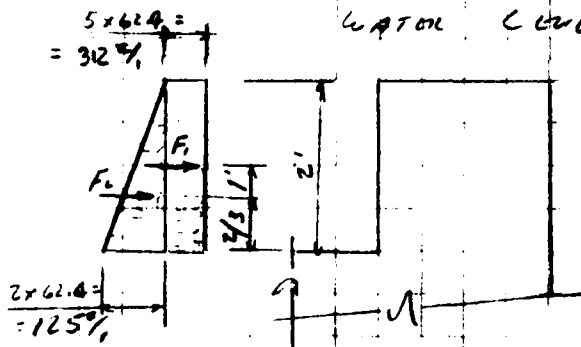
\*  $l_n + d = 15 + 7 = 22 < 5$  ✓

\*\*\*  $\phi_c = .55 \times 2(f'_c)^{1/2} = 60 \text{ PSI}$  ✓

$V_u = 21,000 + 2 \times 7 \times 120 = 10,700 \text{ LBS}$  ✓

$\therefore$  REBAR REQUIRED

VERT. STALL IN CRIP. SILL



$F_1 = 312 \times 2 = 624 \text{ LBS}$  ✓  
 $F_2 = 125 \times 2 = 250 \text{ LBS}$  ✓

$M_{um} = 624 \times 1 + 250 \times \frac{2}{3} = 707 \text{ LBS-FT}$  ✓

$d = 24 - 4 - 2 = 18 \text{ FT}$  ✓

$A_s = 707 \times 12 \div 20 \times 891 \times 18 = .029 \text{ IN}^2$  ✓  
 #5 @ 18" (As = .21 IN<sup>2</sup>)

SEE SH 23, TEMP REBAR GOVERNS  
 $\therefore$  USE #5 @ 6" ✓

SMALLER OF PREVIOUSLY OK

HOWARD NEEDLES TAMMEN & BERENSON CONSULTING ENGINEERS

# HNTB

CALCULATIONS FOR

MADE BY L.D. DATE 4/15/75 JOB NO. 4204  
CHECKED BY J.K.T. DATE 4/18/75 SEC. NO. 5-23  
SHEET NO. 5-23

Cox Steel, Thickness 11 Y

CHUCK TEMP & SURFACE KEYS

REF EM 1119-2-2103 1/2 S PMS 126(4)

$$A_s/F_m = .004 \times 4 \times 7.5 \times 1.44 \div 2 = 9.3 \text{ ML/Em}$$

$$9.3 \div 7.5 = .59 \text{ } \frac{1}{2} \text{ } \therefore \text{USE } 506 \text{ (} \phi = .62 \text{ )}$$

TEMP REIN PARALLEL TO RESTRAINED EDGE  
ALSO @ .40% V

$\therefore$  USE 506 IN VERTICAL DIRECTION

CHUCK

$$A_s/F_m = \frac{.004}{2} \times 24 \times 12 = .58 \text{ } \frac{1}{2} \text{ }$$

HOWARD NEEDLES TAMMEN & BERGENDOFF CONSULTING ENGINEERS

# HNTB

CALCULATIONS FOR

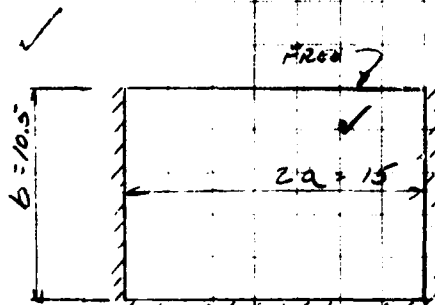
Cor GLEN, ITHACA, N.Y.

MADE BY W.D. DATE 4/18/75 JOB NO. 2255  
 CHECKED BY 7K8 DATE 4/18/75 SEC. NO.         
 SHEET NO. 5-22

End Site (Western)

CASE a & b as noted on SHEET NO. 9  
 (H, N, T & B) WILL BE INVESTIGATED FOR MAX  
 MOM.

REF. ENG. TRANSMISSION 112 27  
 "NORM & RICHARD" REED, PENTON  
 U.S. DEPT. OF I.S.T.  
 BUREAU OF RECREATION

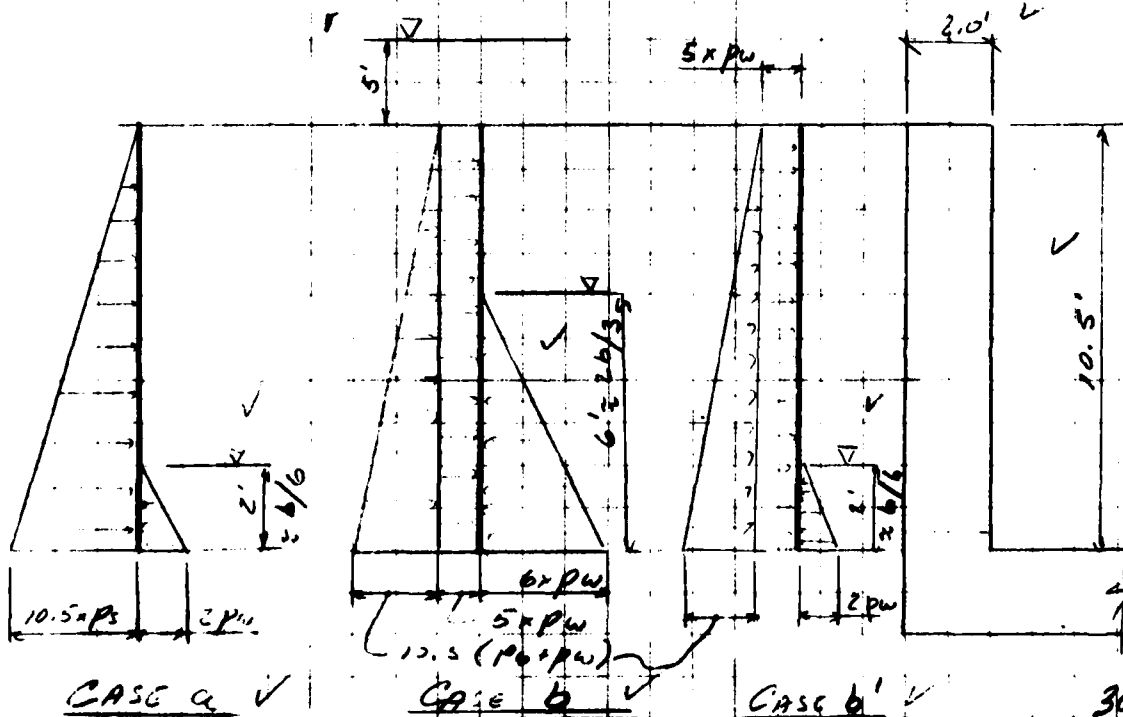


$a = 7.5$

$b = 10.5$

$a/b = 7.5/10.5 = .71$

USE 3/4 FOR  $a/b$



HOWARD NEEDLES TAMMEN & BERGENDOFF CONSULTING ENGINEERS

# HNTB

CALCULATIONS FOR

Cox Green, Indiana, 117

MADE BY C.D.

DATE 9/20/71

JOB NO. 9104

CHECKED BY J.H.T.

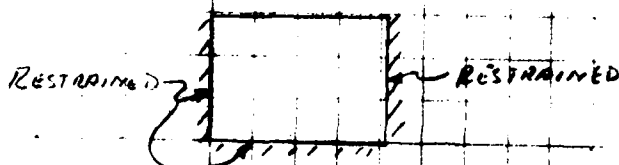
DATE 4/18/75

SEC. NO.

SHEET NO. 5-25

FIND MAX. MOMENT & LENGTH REINFORCED  
DUE TO TRUCK & SIDEWALK

REF. LET 111-2-2103 PG 5 b(4)



USE MEMBERS RESTRAINED TO CRACKING EDGES  
FOR BOTH HORIZ. & VERT. STRESS IN  
FACES. (CONCRETE)

$$R_s / f_{max} = \frac{0.04}{2} \times 24'' \times 12'' = 0.58 \text{ in}^2$$

$$\therefore \#7 @ 12'' \text{ HORIZ. (} R_s = 0.60 \text{ in}^2 \text{)}$$

SIZE FOR MAX. MOMENT THAT WILL BE  
TAKEN WITH MIN. REINFORC.

$$d = 24 - 2 - 1 = 15''$$

$$M_{max} = A_s \times f_s \times d \times \frac{1}{12}$$

$$M = 1.60 \times 20 \times 891 \times 15 \div 12 = 16.9 \text{ in}^2$$

$$\therefore \text{DESIGN FOR MOM} > 16.9 \text{ in}^2$$

HOWARD NEEDLES TAMMEN & BERENSON OFF CONSULTING ENGINEERS

# HNTB

CALCULATIONS FOR

Box Girder, I-10, 10' 0" x 10' 0"

MADE BY C.D. DATE 11/12/77 JOB NO. 4100  
 CHECKED BY J.H.J. DATE 4/18/78 SEC. NO. 5-26  
 SHEET NO. 5-26

$$\begin{aligned} P_s &= 66.5 \frac{\text{K}}{\text{ft}} \checkmark \\ P_L &= 35.3 \frac{\text{K}}{\text{ft}} \checkmark \\ P_u &= 68.9 \frac{\text{K}}{\text{ft}} \checkmark \end{aligned}$$

See H.L.T.F.R. Sta 1+00  $\checkmark$

CASE 2

- (1) FIND MAX MOM. (Pos + Neg) FOR VERT STEEL  $\checkmark$
- (2) NEGLECT WIND IN BOX SINCE IT WILL BE LESS  $\checkmark$
- MOMENTS (MORE CONSERVATIVE)

FIG 2

$$\begin{aligned} \#4 @ 10' & 10.5^2 \times .0065 \times 10.5 \times (.0584) \\ @ 4' & \times (-0.143) \end{aligned}$$

$$\begin{array}{rcl} & +M_x & -M_x \\ & 4.50^{10'} & -1.10^{10'} \\ \hline & +4.50^{10'} & -1.10^{10'} \end{array}$$

Box Moments are 16.5  $\frac{\text{K}}{\text{ft}}$   $\checkmark$

$\therefore$  Tensile & Shear Steel Gov.

- (1) FIND MAX MOM. (Pos + Neg) FOR VERT STEEL  $\checkmark$
- (2) NEGLECT WIND IN BOX SINCE IT WILL BE LESS  $\checkmark$
- MOMENTS (MORE CONSERVATIVE)

FIG 2

$$\begin{aligned} \#4 @ 8' & 10.5^2 \times .0065 \times 10.5 \times (.0434) \\ @ 1' & \times (-0.0214) \end{aligned}$$

$$\begin{array}{rcl} & +M_x & -M_x \\ & 3.33^{10'} & -1.65^{10'} \\ \hline & +3.33^{10'} & -1.65^{10'} \end{array}$$

Box Moments are 16.5  $\frac{\text{K}}{\text{ft}}$   $\checkmark$

$\therefore$  Tensile & Shear Steel Gov.  $\checkmark$

# HNTB

CALCULATIONS FOR

COR GUICH, ITHACA, NY

MADE BY C.D. DATE 2/11/25 JOB NO. 9104  
CHECKED BY J.H.J. DATE 4/16/75 SEC. NO.         
SHEET NO. 5-22

CASE b ✓

- (1) FIND MAX MOM. (POS & NEG) FOR VERT STEEL. ✓  
(2) NEGLECT 6xP4 LOAD SINCE IT WILL BE COVERED. ✓  
143M. (CON. CONTINUED)

FIG 2 ✓

✓ ✓ ✓ ✓  
#1 3x/1 ✓  $5 \times .0024 \times 10.5^2 \times (.1212)$  ✓  
#1 6x/1 ✓ ✓ ✓  $\times (- .0245)$  ✓  
#1 6x/1 ✓  $10.5^3 \times 0.0977 \times (.0584)$  ✓  
#4 6x/1 ✓  $\times (- .0139)$  ✓

✓ ✓  
+M<sub>x</sub> -M<sub>x</sub>  
4.17 ✓  
—  
6.61 ✓  
—  
12.28<sup>100</sup> ✓ 2.41<sup>100</sup> ✓

Both Moments Less Than 16.5<sup>100</sup> ✓  
∴ Tension & Str. Ratio Good ✓

- (1) FIND MAX MOM. (POS & NEG) FOR VERT STEEL ✓  
(2) NEGLECT 6xP4 LOAD SINCE IT WILL BE COVERED ✓

FIG 2 ✓

✓ ✓ ✓ ✓  
#1 6x/1 ✓  $5 \times .0024 \times 10.5^2 \times (.1788)$  ✓  
#1 6x/1 ✓ ✓ ✓  $\times (- .0807)$  ✓  
#4 6x/1 ✓  $10.5^3 \times 0.0977 \times (.0406)$  ✓  
#4 6x/1 ✓  $\times (- .0214)$  ✓

✓ ✓  
+M<sub>x</sub> -M<sub>x</sub>  
6.15 ✓  
—  
4.59 ✓  
—  
10.74<sup>100</sup> ✓ -2.42<sup>100</sup> ✓  
-5.20<sup>100</sup> ✓

Both Moments Less Than 16.5<sup>100</sup> ✓  
∴ Tension & Str. Ratio Good ✓

CASE b' ✓

SINCE WATER IN BOX WAS NOT NEGLECTED  
IN CASE b, RESULTS FOR CASE b' WILL BE SAME AS CASE b ✓ 33

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# HNTB

CALCULATIONS FOR

COY GREEN, TAMMEN, 11.4

MADE BY (C.D.)

DATE 4/14/55

JOB NO. 9109

CHECKED BY J.K.

DATE 4/18/55

SEC. NO.

SHEET NO. 5-28

FIELD MAX. SHEAR

CASE 6 WITH DISBURSED JOINT (NO JOINTS  
6 x 10 ft)

$$F_{10} \text{ @ } 4/1 \quad .0715 \times 5 \times .0624 \times 10.5 = 2.20 \%$$

$$24 \text{ @ } 0/1 \quad .0055 \times 10.5^2 \times .0977 = 6.37 \%$$

SEC. 54. 15 (H117 + B) FOR MAXIMUM & R.O.F.

W.C. 60 pps

$$V_{M1} = .124 \quad V_{M2} = 4 = .104 \times 6.970 = 24.4$$

$$= 28.5 \text{ pps} < 60 \text{ pps}$$

SINCE REPAIR IS GOOD BY THE TIME &  
SHRINKAGE REQUIREMENTS IT WILL BE OK  
TO SPLICE BARS AT SAME LOCATION

$$\text{SPLICE LENGTH } L_D = 17.5 \times 1.7 = 29.75 \text{ SAY } 30"$$

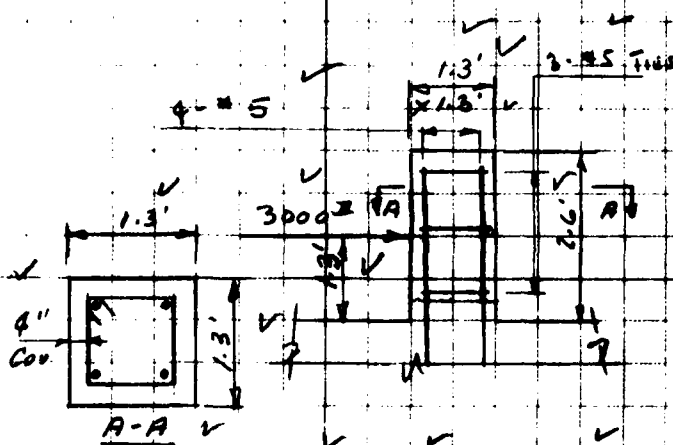
\* SEC. NO. 12 (H117 + B) \*  
\* ACT. STD 318-71 PG. 18 CONC. & FIXTURE

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CALCULATIONS FOR Coy Glen ITHACA N.Y.

## Block Design



DAMI FIRMING INC D-1

300<sup>2</sup> from SH 11-3

Bx H. N. T. G. B.

$$M = 3000 \times 1.3 = 3900' = 3.9'$$

$$\text{say } d = 1.3' \times 12 + 9'' - .62 - .31 = 10.47'$$

Ans:  $3.9 \times 12 \div 20 \times .891 \times 10.67 = 0.25 \text{ m}$

$\omega_{sc} = \frac{Q}{C} = \frac{0.62}{0.001} = 620$

Circle      Square

$$v_c = .55 \times 2(f'c)^{1/2} = 60 \text{ psi (same as max } v_c)$$

NCT 319-71 Pg 36 4 11.2.1

$$N_m = \bar{U} - K_b G_{\text{end}} = 300 - 1 \times 1.8 \times 12 \times 10.68$$

$$= 18.0 \text{ psi} < 60 \text{ psi}$$

Use 1/2" (T10)

# HNTB

CALCULATIONS FOR

COY GLEN, ITHACA, N.Y.

MADE BY L.D. DATE 4/10/75 JOB NO. 4209  
CHECKED BY J.K.G. DATE 4-18-75 SEC. NO.   
SHEET NO. 5-50

CHECK UPGRADE ✓

TRY SLAB 2'-0" THICK ✓

SINCE SLAB IS RIGID ASSUME ENTIRE D.L. OF  
BOX IS SPACED UNIFORMLY OVER CENTER  
AREA BETWEEN SLAB & SOIL. ✓

ALL CALCULATIONS FOR UPGRADE BASED ON  
STREET W.D. 1. ✓

USE AVE. DEPTH OF WATER IN DETERMINING  
BUOYANT FORCE. ✓

POOL DEPTH OF 2' IN BOX (CONSERVATIVE) ✓

CONSULTING ENGINEER  
HNTB  
1000 MARKET STREET, SUITE 200, NEW YORK, N.Y. 10036  
TELEPHONE (212) 512-2000  
FACSIMILE (212) 512-2001  
TELETYPE (212) 512-2002

# HNTB

CALCULATIONS FOR

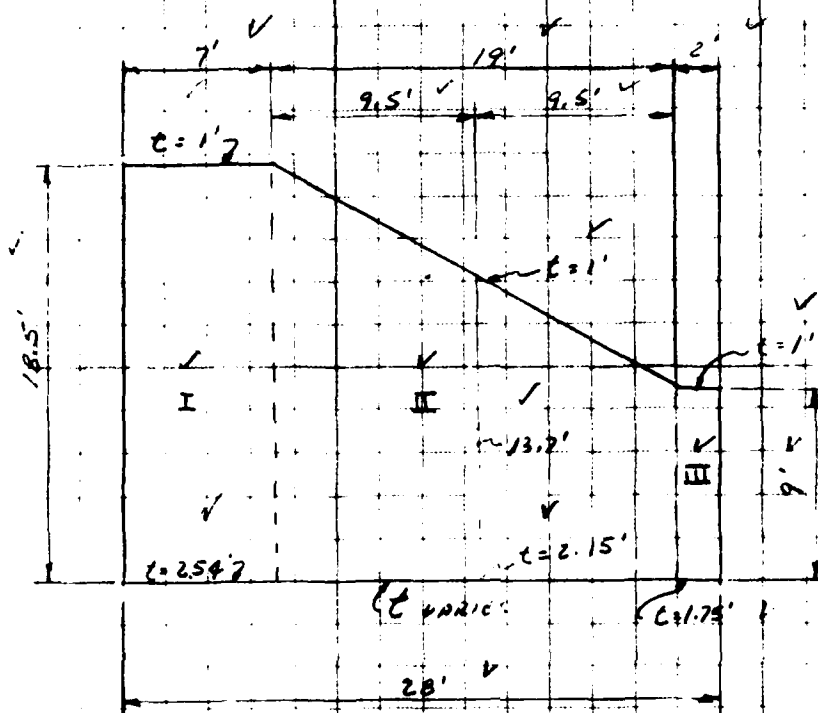
JOY GREEN, ITHACA, N.Y.

MADE BY L.D.  
CHECKED BY T.K.T.

DATE 9/10/75  
DATE 4/18/78

JOB NO. 4204  
SEC. NO.  
SHEET NO. 5-31

BOX WALLS



C = Thickness

$$Vol I = (1 + 2.54) \times \frac{1}{2} \times 18.5 \times 7 = 229 \text{ Ft}^3$$

$$Vol II = \frac{1}{6} \left[ (1 + 2.54) \times 18.5 + 4(1 + 2.15) \times 13.2 + (1 + 1.75) \times 9 \right] = 416 \text{ Ft}^3$$

$$Vol III = (1 + 1.75) \times \frac{1}{2} \times 22.8 = 25 \text{ Ft}^3$$

$$\Sigma 1 \text{ WALL} = 670 \text{ Ft}^3$$

$$\Sigma 2 \text{ WALL} = 2 \times 670 = 1340 \text{ Ft}^3$$

End Sides

$$Vol up = 2.0' \times 10.5' \times 15' = 315 \text{ Ft}^3$$

$$Vol down = (2 + 3) \times 2 \times 15' = 150 \text{ Ft}^3$$

End Sides

$$= 465 \text{ Ft}^3$$

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# HNTB

CALCULATIONS FOR

COY GREEN, TIER 1, 9

MADE BY

C.D.

DATE

4/12/75

JOB NO.

4254

CHECKED BY

J.K.J.

DATE

4.19.75

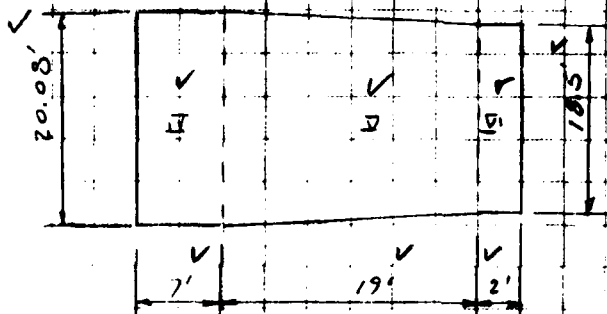
SEC. NO.

SHEET NO. 5-32

Blocks

$$Vol = 5 \times 1.3' \times 1.3' \times 1.6' = 22 \text{ Ft}^3$$

SLAB



$$Area \text{ L} = 7 \times 20.08 = 140.6 \text{ Ft}^2$$

$$" \text{ M} = (20.08 + 18.5) \times \frac{1}{2} \times 19 = 366.5 \text{ Ft}^2$$

$$" \text{ R} = 2 \times 18.5 = 37.0 \text{ Ft}^2$$

$$544.1 \text{ Ft}^2$$

$$Vol = 544.1 \times 2.5 = 1360 \text{ Ft}^3$$

WATER (2' Deep)

$$WT = .0624 \times 2' \times 15' \times 24' = 45 \text{ k}$$

$$\pm Conc WT = .15 \times (1360 + 465 + 22 + 1360) = 478 \text{ k}$$

$$\therefore \text{TOTAL DOWNWARD LOAD} = 523 \text{ k}$$

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# HNTB

MADE BY C.D. DATE 4/10/75 JOB NO. 4204  
CHECKED BY J.H.Z. DATE 4/19/75 SEC. NO. 5-33  
SHEET NO. 5-33

CALCULATIONS FOR

COY GLEN, ITHACA, N.Y.

## BUOYANT FORCE

FOR HEAD 5.0 SM 1.8 (MIN. T.G. L.)  
FTG THICKNESS

$$H = 11.3' + 2.5' = 13.8' \checkmark$$

$$P = 13.8 \times 0.624 = 8.61 \text{ k/ft} \checkmark$$

$$\text{BUOYANT LOAD} = 540.1 \times 8.61 = 468 \text{ k} \checkmark$$

$$\text{MIN. F.S. AGAINST UPLIFT} = 523 \div 468 = 1.12 \checkmark$$

F.S.  $\checkmark$  WOULD BE GREATER THAN 1.12 SINCE  
FRICTION BETWEEN WALL AND EARTH, AND  
VERTICAL LOAD ON SLOPING OUTSIDE FACE  
OF BOX WALL HAS NOT BEEN INCLUDED.  $\checkmark$

CONSULTING ENGINEERS

HOWARD NEEDLES TAMMEN & BERGENDOFF

# HNTB

CALCULATIONS FOR

POY GICH, ITANCA, N.Y.

MADE BY

C.V.

DATE

4/11/75

JOB NO.

9204

CHECKED BY

J.K.T.

DATE

4/19/75

SEC. NO.

SHEET NO. 5-34

## BOTTOM SLAB DESIGN ✓

SINCE SLAB IS RIGID ASSUME D.C. OF BOX WILL RESULT IN A UNIFORM UPWARD EARTH PRESSURE. ✓

### LOADING CASE I ✓

~ BOX EMPTY NO HYD. IMP OR BUOYANT FORCES. (NEG. SLAB MOM) ✓

### LOADING CASE II ✓

~ HYD. IMP AND BUOYANT FORCES. (POS. SLAB MOM) ✓

✓  
SLAB THICKNESS AND HEIGHT OF WALL WILL BE SELF CANCELING IN THE DESIGN OF SLAB. ✓

HOWARD NEEDLES TAMMEN & BERGENCOFF CONSULTING ENGINEERS

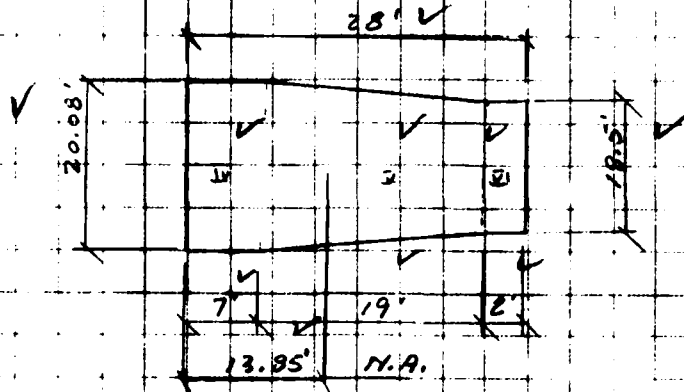
# HNTB

CALCULATIONS FOR

COY GLEN, LINDAIA, N.Y.

MADE BY C.D. DATE 9/11/75 JOB NO. 4204  
 CHECKED BY J.K.T. DATE 6/19/75 SEC. NO. 5-35  
 SHEET NO. 5-35

FIELD SECT. MOM. OF SCAB



	AREA	C	AC	$\bar{x}$	$A\bar{x}$	$I_{oo}$
I	140.6	3.5	492	10.35	15,061	574
II	366.5	16.5	6047	2.65	2,574	11,020
III	320	27.0	999	13.15	4,398	12
$\Sigma$	544.1		7538		24,033	11,606

$$\bar{x} = 7538 \div 544.1 = 13.85'$$

\* AISC 6th Ed. 1943 Pg 6-25

$$C = \frac{12}{3} (2 \times 20.08 + 18.5) \div (20.08 + 18.5) = 9.63'$$

$$I_{oo} = \frac{12^3}{36} (20.08^2 + 4 \times 20.08 \times 18.5 + 18.5^2) \div 36 (20.08 + 18.5) + 11,020$$

$$S.M. unsymmetrical = 35,639 \div 13.85 = 2573 \text{ FT}^3$$

$$S.M. symmetrical = 35,639 \div 14.15 = 2519 \text{ FT}^3$$

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# HNTB

CALCULATIONS FOR

MADE BY CV DATE 1/11/75 JOB NO. 912 J  
 CHECKED BY JR7 DATE 4/19/75 SEC. NO.   
 SHEET NO. 5-36

LOADING CASE 1 (see SH 113 34)

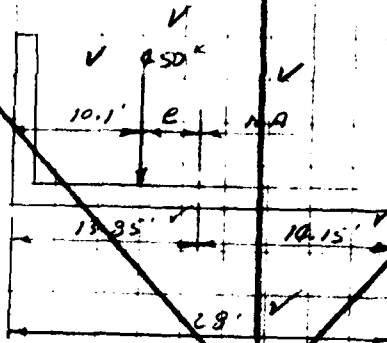
TOTAL WT OF COILS = WT OF 4 COILS  
 $= 273 - .15 \times 1363 = 274^k$  (see SH 113 34)

UNIFORM PRESS. DUE TO W.C. =  $274^k \div 544.1^2 = .504^k$

LOADING CASE 2 (see SH 113 34)

10" H. 20" W. 48" D. 11" L. 12" T

$= 6.0 \times 5 \times 15 = 450^k$   
 (see SH 113 34)



$C = 18.85 - 10.1 = 8.75'$

$171 - 450 = 225 = 1658^k$

UPPER CASE  $P = \frac{450}{544} + \frac{1658}{2519} = 1.48^k/12$  NET SILE  
 LOWER CASE  $P = \frac{450}{544} - \frac{1658}{2519} = 0.16^k/12$  PRESSURE

TOTAL UNIFORM PRESS. =  $523 - 468 = 55^k$   
 $P = 55 \div 544 = .101^k/12$

Deletion of

HOWARD NEEDLES TAMMEN & BERGENDOFF  
 CONSULTING ENGINEERS

Subject Coy Glen

Page \_\_\_\_ of \_\_\_\_ pages.

Sheet 36A

Computation of

Computed by AJA

Checked by RSG

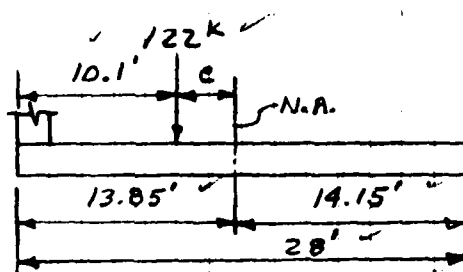
Date 1/12/76

LOADING CASE II

Total Downward Loading From Impact =

$$1.63 \times 5 \times 15 = 122.25 \text{ K}$$

see P. 12BA



$$e = 13.85 - 10.1 = 3.75'$$

$$M = 122 \times 3.75 = 458 \text{ K}'$$

$$\text{Upstream } p = \frac{122}{544} + \frac{458}{2573} = .402 \text{ K/s.F.}$$

$$\text{Downstream } p = \frac{122}{544} - \frac{458}{2573} = .046 \text{ K/s.F.}$$

Find Slab Load with Buoyant Condition and 2' pool of water in box.

$$\text{Total Downward load } \overset{P.38}{523 \text{ K}} - \overset{P.39}{468 \text{ K}} = 55 \text{ K}$$

$$p = 55 \div 544 = .101 \text{ K/s.F.} \downarrow$$

# HNTB

CALCULATIONS FOR

MADE BY C.V. DATE 7/17/75 JOB NO. 4204  
 CHECKED BY 7K7 DATE 8.19.75 SEC. NO. 5-37  
 SHEET NO. 5-37

Find Downward Load  $W_d$  to 2.5' of  
 soil + 2' of water

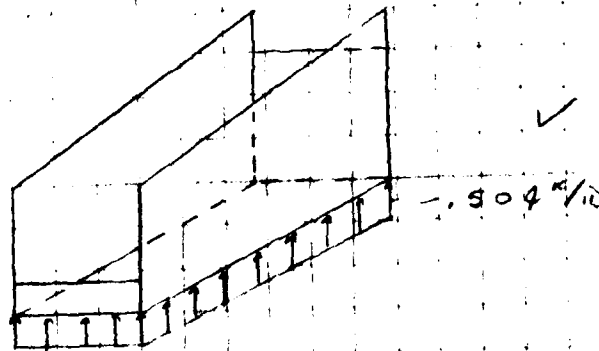
$$\begin{aligned} 2 \times .0624 &= .125 \text{ k} \\ 1.5 \times .15 &= .375 \text{ k} \\ .500 \text{ k} & \end{aligned}$$

1. Downward Load for 2.5' of soil + 2' of water

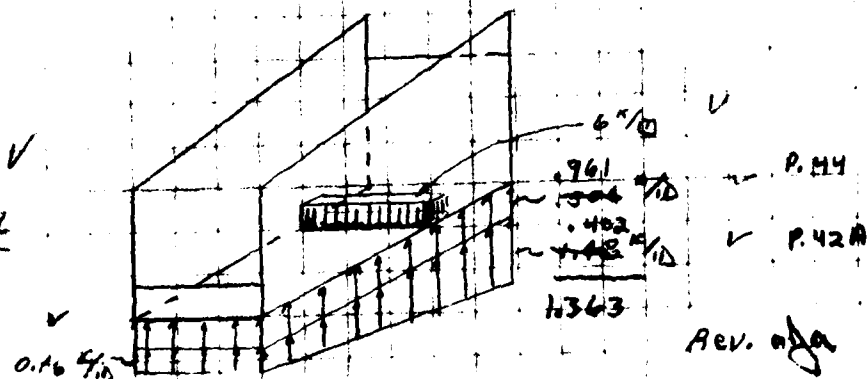
$$-.101 \frac{\text{k}}{\text{ft}} + .502 \frac{\text{k}}{\text{ft}} = 0.399 \frac{\text{k}}{\text{ft}}$$

By Inspection, Imp. Corroding, High Yield. For  
 307700 Steel.

LOADING CASE I



LOADING CASE II



HOWARD NEEDLES TAMMEN & BERGENDOFF CONSULTING ENGINEERS

### CALCULATIONS FOR

MADE BY C.D. DATE 5/5/75 JOB NO. 4104  
CHECKED BY JKF DATE 5-8-75 SEC. NO. \_\_\_\_\_  
SHEET NO. 5-37A

Сыг. 1100. Итого 1100.

Final Max Size Pressure Under Box

MAX. SOIL PRESSURE WILL OCCUR AT  
2' POOL OF WATER IN BOX. IMPACT  
AND 12 MON - BUDGET CIRCUMSTANCES.

D.C. of 120 + 2' GATE = 523' + 500' = 1.361' <sup>sec 34 12 32</sup>  
 From condition, upstream gate = ~~500' + 1.48' = 1.48' <sup>sec 34 32</sup>~~  
 → P. 42A  
 Rev. also 1.363' KS

Allowable bearing: 2 Tons/SF

Factor of Safety :  $\frac{4.0}{1.363} = 2.9$  ok

**HOWARD NEEDLES TAMMEN & BERGENDOFF**  
CONSULTING ENGINEERS

# HNTB

CALCULATIONS FOR

COY GLEN, ITHACA, N.Y.

MADE BY L.D. DATE 4/18/75 JOB NO. 4204  
 CHECKED BY J.R. DATE 4.19.75 SEC. NO.   
 SHEET NO. 5-38

TO OBTAIN DESIGN MAX 1537 5000  
 "SE" "REINFORCED CONCRETE DESIGNER'S HANDBOOK"  
 BY CHAS. C. REYNOLDS 6th EDITION ✓

SEE HNTB  
 SM NO 384  
 P 38

{ REF. PG 206 & 207, FOR UNIFORM LOAD ✓  
 { REF. PG 214 & 215, FOR STEEL LOAD OF 6% ✓

FOR THE TRIANGULAR LOAD IN CRACKING  
CASE I, ASSUME AN AVERAGE UNIFORM  
PRESSURE ✓

LOADING CASE I (SEE SM NO 37)

CORNERS MOD DOWN

TABLE 38 {  $R = 24' \div 15' = 1.6$  ✓  
 $K_1 = 0.63$  ✓  $K_2 = 0.09$  ✓

(TRANS. MOM)  $M_R = K_1 W L^2 \div 8 =$  ✓  
 $= -0.63 \times (.504) \times 15^2 \div 8 = -8.9'k$  ✓

(LONG. MOM)  $M_L = K_2 W L^2 \div 8 =$  ✓  
 $= -0.09 \times (.504) \times 24^2 \div 8 = -3.3'k$  ✓

HOWARD NEEDLES TAMMEN & BERGENDOFF CONSULTING ENGINEERS

# HNTB

CALCULATIONS FOR

MADE BY \_\_\_\_\_ DATE \_\_\_\_\_ JOB NO. 9204  
CHECKED BY \_\_\_\_\_ DATE \_\_\_\_\_ SEC. NO. \_\_\_\_\_  
SHEET NO. 5-38A

COY GIEM, ITHACA, NY

HOWARD NEEDLES TAMMEN & BERGENDOFF CONSULTING ENGINEERS

## REINFORCED CONCRETE DESIGNER'S HANDBOOK

BY  
CHAS. E. REYNOLDS  
B.Sc.(Eng.), M.Inst.C.E.

SIXTH EDITION



PUBLISHED BY  
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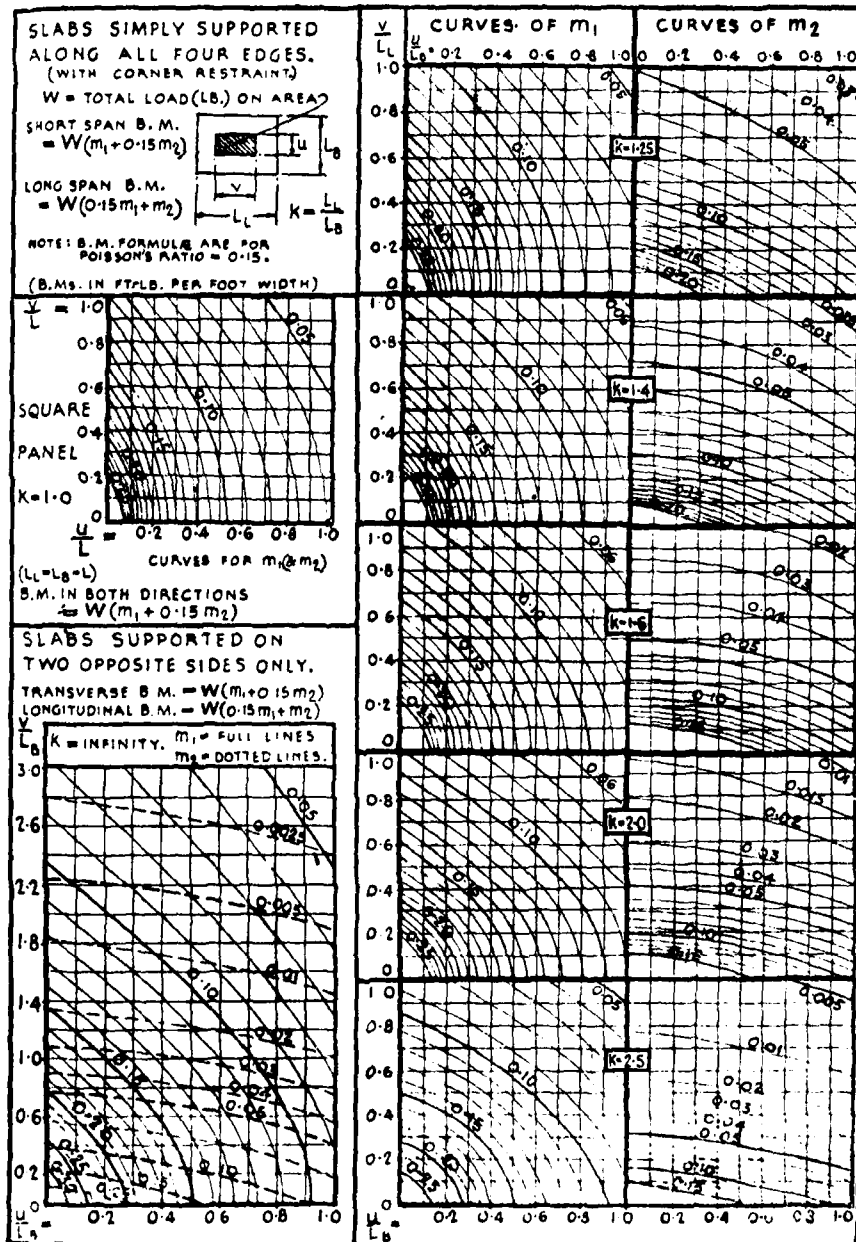
# HNTB

CALCULATIONS FOR

Cox Glen, ITHACA, N.Y.

MADE BY \_\_\_\_\_ DATE \_\_\_\_\_ JOB NO. 2202  
 CHECKED BY \_\_\_\_\_ DATE \_\_\_\_\_ SEC. NO. \_\_\_\_\_  
 SHEET NO. 5-3813

SLABS SPANNING IN TWO DIRECTIONS: RECTANGULAR PANELS. TABLE 42.  
 CONCENTRIC CONCENTRATED LOADS.



NOTE.—See note regarding square panels on page 214.

## 206 REINFORCED CONCRETE DESIGNER'S HANDBOOK

### SLABS SPANNING IN TWO DIRECTIONS.

**Notation.**—The symbols used in the following and in Tables 38, 39, 41, 42 and 43 are as follows. (The corresponding symbols used in B.S. Code No. 114, where different, are given in brackets.) The symbols used in Tables 40 and 44 are given in the respective tables.

$W$  = total load (lb.) on the slab and equal to  $wL_B L_L$  for a completely-loaded panel, and equal to  $wuw$  for a partially-loaded panel;  $w$  = uniformly-distributed load (lb. per sq. ft.).

$L_B (l_B)$  and  $L_L (l_L)$  = short and long spans (ft.) respectively,  $k = \frac{L_L}{L_B}$ .

$M_B$  and  $M_L$  = maximum bending moments at the midspan of the short and long spans respectively;  $M_{BA}$  and  $M_{BC}$  = bending moments at supports A and C respectively of the short span;  $M_{LD}$  and  $M_{LE}$ , the bending moments at supports D and E respectively of the long span. Bending moments are in ft.-lb. per foot width.

$K_B$  and  $K_L$  = bending-moment reduction factors for short and long spans respectively, corners not held down;  $K'_B$  and  $K'_L$  = corresponding factors with corners held down.

$m_B (= \beta_B)$  and  $m_L \left[ -\beta_L \left( \frac{l_L}{l_B} \right)^2 \right]$  = coefficients for positive bending moments on short and long spans respectively, with corners held down;  $m'_B$  and  $m'_L$  = corresponding coefficients for negative bending moments.  $m_{BO} (= \alpha_B)$  and  $m_{LO} \left[ = \alpha_L \left( \frac{l_L}{l_B} \right)^2 \right]$  = coefficients for positive bending moments on short and long spans respectively with corners not held down.

**Rectangular Panels Freely Supported along All Edges with Uniformly-distributed Load.**—For a rectangular panel that is freely supported along all four edges in such a manner that the corners are free to lift, the Grashof and Rankine method is applicable and the bending moment reduction coefficients are  $K_B = \frac{k^4}{k^4 + 1}$  and  $K_L = 1 - K_B$ . The midspan bending moments per foot width  $M_B$  and  $M_L$  are calculated from the formulae in Table 38. The usual limit of application of this method is when the length of the panel is equal to twice the breadth, that is when  $k = 2$ . Beyond this limit the slab is considered to span across the short span only, the bending moment per foot width then being  $\frac{wL_B^2}{8}$ .

For the condition "corners not held down", the bending-moment coefficients in the B.S. Code correspond to  $m_{BO}$  and  $m_{LO}$  in Table 39.

In cases near the limit of  $k = 2$ , it is necessary to ensure that the amount of reinforcement in the long direction is not less than the minimum amount of distribution bars required.

For panels that are freely supported along all four edges but with the corners prevented from lifting, the corresponding coefficients  $K'_B$  and  $K'_L$  in Table 38 conform to a more exact analysis but with Poisson's ratio equal to zero.

The bending moments at midspan based on Dr. Marcus's method are the midspan bending moments calculated by the Grashof and Rankine method multiplied by a factor  $C$ ; for a slab freely supported along all four edges  $C = 1 - \frac{5}{6} \frac{k^4}{(1 + k^4)}$ ; the midspan bending moments per foot width are  $M_B = CK_B \frac{wL_B^2}{8}$  and  $M_L = \frac{M_B}{k^2}$ .

The resultant bending moments by the method of Dr. Marcus and the exact theory (with Poisson's ratio equal to zero) are almost identical. If Poisson's ratio is assumed to be 0.15, the midspan bending moments per foot of width are  $M_B = \frac{wCK_B L_B^2}{8} \left( 1 + \frac{0.15}{k^2} \right)$  and  $M_L = \frac{wCK_B L_B^2}{8} \left( 0.15 + \frac{1}{k^2} \right)$ . Alternatively the appropriate coefficients can be obtained

from the curves in Table 42 for  $\frac{u}{L_B} = \frac{v}{L_L} = 1$  for a slab completely covered with a load of intensity  $w = \frac{W}{L_B L_L}$ . The bending-moment coefficients given in the B.S. Code for this case correspond to  $m_B$  and  $m_L$  in the top left-hand corner in Table 39.

**Rectangular Panels Fixed along Four Sides with Uniformly-distributed Load.**—If a panel is completely fixed along all four sides, the bending moments are as follows.

Short span: Midspan  $M_B = +0.8M_{BA}$ ; Support  $M_{BA} = -K'_B \frac{wL_B^2}{8}$ .

Long span: Midspan  $M_L = +0.8M_{LD}$ ; Support  $M_{LD} = -K'_L \frac{wL_L^2}{8}$ .

where  $K'_B$  and  $K'_L$  are as in Table 38. (See also B.S.-code method on page 208.)

(Continued on page 208.)



SLABS SPANNING IN TWO DIRECTIONS: RECTANGULAR PANELS. -TABLE 38.  
UNIFORMLY-DISTRIBUTED LOAD.

RATIO OF SPANS	CONDITION ALONG FOUR EDGES					
	FREE CORNERS NOT HELD DOWN:		FREE OR FIXED CORNERS HELD DOWN:		FIXED (OR MARCUS)	
	K	K <sub>B</sub>	K <sub>L</sub>	K <sub>B</sub> '	K <sub>L</sub> '	C'
1-0	0-50	0-50	0-30	0-30	0-861	
1-05	0-55	0-45	0-33	0-27	0-862	
1-1	0-59	0-41	0-36	0-24	0-864	
1-15	0-64	0-36	0-39	0-22	0-866	
1-2	0-68	0-33	0-42	0-19	0-871	
1-25	0-71	0-29	0-45	0-17	0-874	
1-3	0-74	0-26	0-48	0-15	0-879	
1-4	0-79	0-21	0-53	0-13	0-888	
1-5	0-84	0-16	0-58	0-11	0-898	
1-6	0-87	0-13	0-63	0-09	0-907	
1-75	0-90	0-10	0-68	0-07	0-919	
2-0	0-94	0-06	0-76	0-05	0-935	
2-5	0-97	0-03	0-87	0-03	0-957	
3-0	0-98	0-02	0-94	0-02	0-970	

UNIFORMLY-DISTRIBUTED LOAD = W L.B. PER SQ. FT.

FREELY-SUPPORTED ALONG FOUR EDGES.

CORNERS NOT HELD DOWN:  $M_B = +K_B \frac{W L^2}{8}$ ;  $M_L = +K_L \frac{W L^2}{8} = \frac{M_B}{K^2}$

CORNERS HELD DOWN:  $M_B = +K_B' \frac{W L^2}{8}$ ;  $M_L = +K_L' \frac{W L^2}{8}$

FIXITY ALONG FOUR EDGES.  
CORNERS HELD DOWN. (OR MARCUS)

$M_B = +C' K_B \frac{W L^2}{24}$ ;  $M_L = +C' K_L \frac{W L^2}{24} = \frac{M_B}{K^2}$

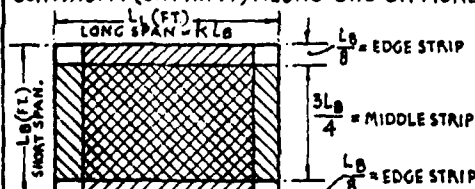
$M_{BA} = M_{BC} = -K_B \frac{W L^2}{12}$

$M_{LD} = M_{LE} = -K_L \frac{W L^2}{12} = \frac{M_{BA}}{K^2}$

CONTINUITY (OR FIXITY) ALONG ONE OR MORE EDGES. (B.S. CODE.)

CONDITIONS:  
CORNERS HELD DOWN.

**TORSIONAL RESISTANCE PROVIDED  
NO REINFORCEMENT REQUIRED IN EDGE  
STRIPS TO RESIST BENDING MOMENT  
PARALLEL TO EDGES OF PANEL.**



$\frac{L_L}{8} \quad \frac{3L_L}{4} \quad \frac{L_L}{8}$  IF  $k > 4.0$   
 EDGE STRIP MIDDLE STRIP EDGE STRIP  
 IF  $k > 4.0$ : WIDTH OF MIDDLE STRIP =  $L_L - L_D$   
 " " EDGE STRIPS =  $0.5 L_A$

BENDING MOMENTS (FT.-LB. PER FOOT) IN MIDDLE STRIPS:-

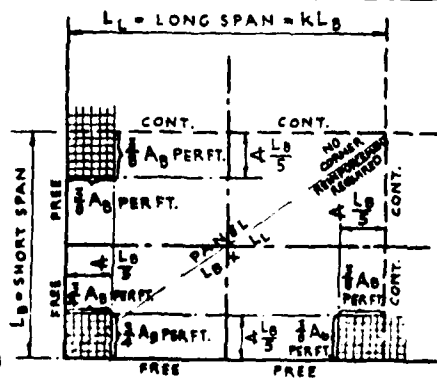
	AT MIDSPAN.	AT CONTINUOUS EDGE.	AT DISCONTINUOUS EDGE. (SLAB MONOLITHIC WITH SUPPORT)
SHORT SPAN:	$+m_B w L_B^2$	$-m_B' w L_B^2$	$-\frac{2}{3}m_B w L_B^2$
LONG SPAN:	$+m_L w L_L^2$	$-m_L' w L_L^2$	$-\frac{2}{3}m_L w L_L^2$

CORNER REINFORCEMENT  
FOR TORSIONAL RESISTANCE.  
(B. S. CODE)

$A_g$  &  $A_L$  = CROSS SECTIONAL AREA (PERFT.)  
OF REINFORCEMENT FOR POSITIVE  
B.M. AT MIDSPAN OF SHORT AND  
LONG SPANS RESPECTIVELY.

$\frac{1}{2}A_c U_f A_g$  = CROSS SECTIONAL AREA (PER FT.)  
OF CORNER REINFORCEMENT IN  
EACH OF TWO LAYERS (ONE NEAR  
TOP FACE OF SLAB; ONE NEAR  
BOTTOM FACE).

IF  $A_L > A_B$  SUBSTITUTE  $\frac{1}{2}A_L \cup \frac{1}{8}A_L$  FOR  $\frac{1}{2}A_B \cup \frac{1}{8}A_B$



NOTE: For values of  $m_p$ ,  $m'_B$ ,  $m_L$  and  $m'_L$  (for calculation of bending moments on middle strips) see Table 39.

## 214 REINFORCED CONCRETE DESIGNER'S HANDBOOK

### SLABS SPANNING IN TWO DIRECTIONS (continued).

**Square Panels** ( $k = 1.0$ ).—The expression in Table 42 that the bending moment in both directions is  $W(m_1 + 0.15m_2)$  applies only if load is over entire panel, or if  $u = v$ .

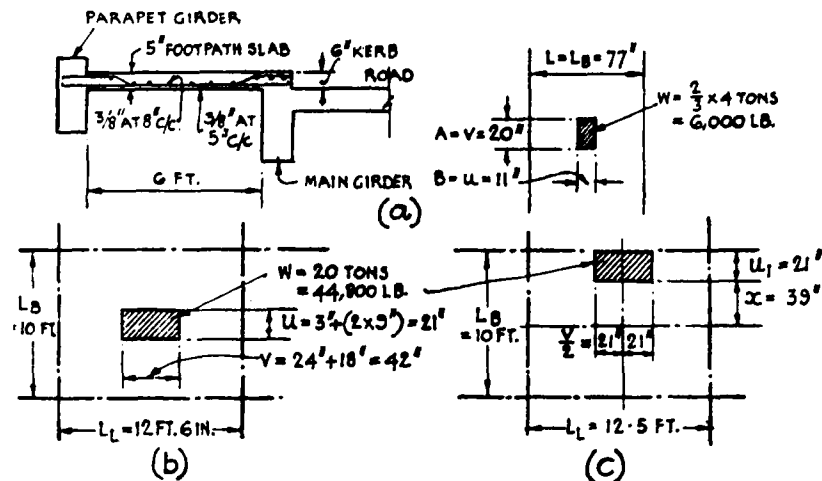
Other conditions.— $u$  and  $v$  can be in either direction;  $m_1$  is the bending moment coefficient in the direction of  $u$ ;  $m_2$  is the coefficient in the direction of  $v$ . Coefficient  $m_1$  is based on  $u$  and  $v$  as selected; for coefficient  $m_2$  reverse  $u$  and  $v$ .

Example.—If  $\frac{u}{L} = 0.8$  and  $\frac{v}{L} = 0.2$ ,  $m_1 = 0.072$ ; for  $\frac{u}{L} = 0.2$  and  $\frac{v}{L} = 0.8$ ,  $m_2 = 0.103$ .

Bending moments.

On span in direction of  $u$ :  $W[0.072 + (0.15 \times 0.103)] = 0.087W$  ft.-lb. per ft.  
On span in direction of  $v$ :  $W[0.103 + (0.15 \times 0.072)] = 0.114W$  ft.-lb. per ft.

**Examples of Panels Supporting Concentrated Loads.**—The following examples illustrate the use of Tables 42 and 43 for slabs supporting a load which is concentrated uniformly over an area less than the entire area of the panel. Notes on these tables are given on page 32.



(a) The footpath of a bridge spans 6 ft. between a parapet girder and a main longitudinal girder, and is monolithic with both girders [diagram (a)]. The live load is either 100 lb. per sq. ft. uniformly distributed, or a load of 4 tons from a wheel the contact area of which is 12 in. by 3 in. (With the latter load the stresses may be increased by 50 per cent.; that is at ordinary working stresses the wheel load can be assumed to be about 6000 lb.) These loads comply with the recommendations of the Ministry of Transport.

(i) Assume a 5-in. slab; total uniformly-distributed load =  $63 + 100 = 163$  lb. per sq. ft. With continuity at both supports, bending moment at midspan and at each support is  $\frac{1}{8} \times 163 \times 6.3^2 \times 12 = 6400$  in.-lb. per ft. width.

(ii) Contact area of 12 in. by 3 in. at the wheel can be increased to 20 in. by 11 in. (Table 6); depth to the reinforcement is about 4 in.

The slab spans mainly in one direction; the curves in the lower left-hand corner of Table 42 apply.  $\frac{u}{L_1} = \frac{11}{77} = 0.143$ ;  $\frac{v}{L_2} = \frac{20}{77} = 0.26$ ;  $m_1 = 0.22$  and  $m_2 = 0.12$ .

Free transverse bending moment =  $6000[0.22 + (0.15 \times 0.12)]12 + (\frac{1}{8} \times 63 \times 6.3^2 \times 12) = 17,150 + 3750 = 20,900$  in.-lb. per ft. width.

Allow for continuity (partial fixity) by reducing the free bending moment due to the dead load by one-third, and that due to the live load by 20 per cent.; the transverse bending

(Continued on page 216.)

# HNTB

CALCULATIONS FOR

CAY GREEN, ITHACA, N.Y.

MADE BY

C.D.

DATE

1/14/55

JOB NO.

9204

CHECKED BY

J.R.T.

DATE

4.2.55

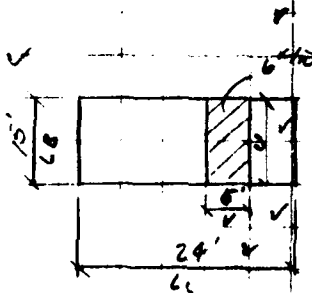
SEC. NO.

SHEET NO. 5-39

LOADING CASE II (SEE SH. NO. 37) ✓

STRIP LOADING = 6 K/ID ✓

$$W = 6 \times 15 \times 5 = 450 \text{ K} \checkmark$$



$$R = 24 \div 15 = 1.6 \checkmark$$

$$U \div C_L = 5 \div 24 = .208 \checkmark$$

$$U \div C_B = 15 \div 15 = 1.0 \checkmark$$

FROM CHART ON TABLE 42  $R=1.6$  ✓ ✓

$$m_1 = .084 \checkmark$$

$$m_2 = .058 \checkmark$$

(AT THE TORSION) ✓

$$M_{12} = W (m_1 + .15 m_2) \checkmark$$

$$= 450 (.084 + .15 \times .058) = +41.7 \frac{\text{K}}{\text{ft}} \checkmark$$

(AT THE MOM) ✓

$$M_{12} = W (.15 m_1 + m_2) \checkmark$$

$$= 450 (.15 \times .084 + .058) = +31.8 \frac{\text{K}}{\text{ft}} \checkmark$$

FOR TRIANGULAR LOAD ✓

$$P = (.16 + 1.42) \times 6 = 0.82 \text{ K/ID} \checkmark$$

HOWARD NEEDLES TAMMEN &amp; BERGENDOFF CONSULTING ENGINEERS

# HNTB

CALCULATIONS FOR Box GLEN, ITHACA, N.Y.

MADE BY C.L. DATE 9/11/25 JOB NO. 1209  
 CHECKED BY J.R.T. DATE 9/21/25 SEC. NO.   
 SHEET NO. 5-40

PROPORTION LAND OF .82% LAND CONTRIBUTING  
 CASE 2 OF 0.5 + 9'  $\frac{1}{2}$  (SEE NO. 39)

(70 M/MI)  $M_{13} = 0.82 \times 5.5 \div .504 = -14.5' \%$

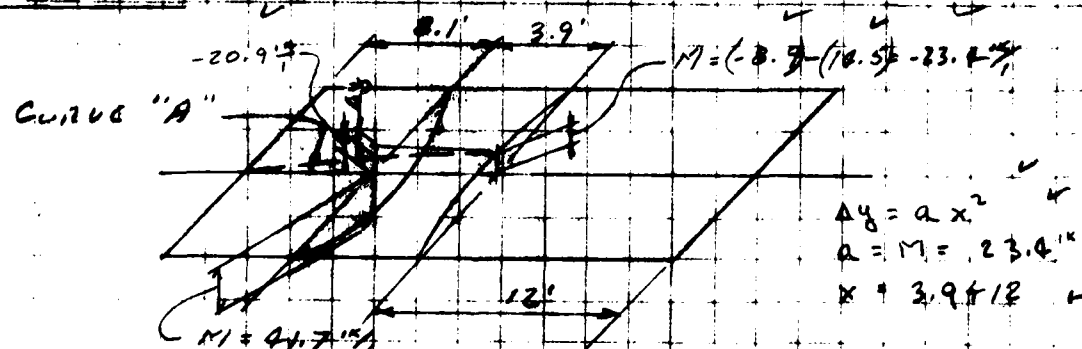
(60 M/MI)  $M_{14} = 0.82 \times 3.3 \div .504 = -5.4' \%$

UNIFORM CD (.504%)  
 TRIANG. CD (10 to 1.88%)  
 $\Sigma$  NEG MOM.

$M_{TRANS}$	$M_{LONG}$
-8.9' $\frac{1}{2}$	-3.3' $\frac{1}{2}$
-14.5' $\frac{1}{2}$	-5.4' $\frac{1}{2}$
-23.4' $\frac{1}{2}$	-8.7' $\frac{1}{2}$

PLAN  $M_{13}$  Fall Box Span Calculations Case II

TRANS MOM.



SINCE POS. & NEG MOM DO NOT OCCUR AT THE SAME LOCATION, CALCULATE NEG. MOM ASSUMING PARABOLIC CURVE 'A' WITH ORIGIN AT  $\Sigma$  OF 23.4'  $\frac{1}{2}$

$\therefore M = -23.4 + \frac{(3.5)^2}{12} = -20.9' \%$

$\therefore \Sigma M = +41.7 \cdot 20.9 = 20.8' \%$



# HNTB

CALCULATIONS FOR

Box Gully, Litchfield, N.Y.

MADE BY LD DATE 4/16/75 JOB NO. 4204  
CHECKED BY JAT DATE 4-21-75 SEC. NO.         
SHEET NO. 5-42

ASSUME #6 BAR

$$d_{TOP LONG.} = 30" - 6" - .375" = 23.625" \quad \text{USE } 23.5"$$

$$d_{TOP TRANS.} = 30" - 6" - .75" - .375" = 22.875" \quad \text{USE } 22.5"$$

$$d_{BOT TRANS.} = 30" - 4" - .75" - .375" = 24.875" \quad \text{USE } 24.5"$$

$$d_{BOT LONG.} = 30" - 4" - .375" = 25.625" \quad \text{USE } 25.5"$$

TRANS. BARS

$$TOP \quad A_s = 8.9 \times 12 \div 20 \times .891 \times 22.5 = .27 \text{ @ } 1 \text{ MIN.} \quad (\#6 @ 12")$$

$$BOT \quad A_s = 20.8 \times 12 \div 20 \times .891 \times 24.5 = .58 \text{ @ } 1 \text{ MIN.} \quad (\#7 @ 12")$$

LONGIT. BARS

$$TOP \quad A_s = 3.8 \times 12 \div 20 \times .891 \times 23.5 = 0.10 \text{ @ } 1 \text{ MIN.} \quad (\#6 @ 12")$$

$$BOT \quad A_s = 24.0 \times 12 \div 20 \times .891 \times 25.5 = 0.64 \text{ @ } 1 \text{ MIN.} \quad (\#5 @ 12") \quad (\#6 @ 12")$$

MIN. REINFORCEMENT

Ref. E17 1110-2-2103

Per 5 (CONCRETE) MIN.

$$MIN. LONG. \& TRANS. = 30 \times 12 \times .0025 \div 2 =$$

$$= .45 \text{ @ } 1 \text{ MIN.}$$

$$\text{USE } \#6 @ 12" (1 MIN.) \quad A_s = .44 \text{ @ } 1 \text{ MIN.}$$

CONSULTING ENGINEERS

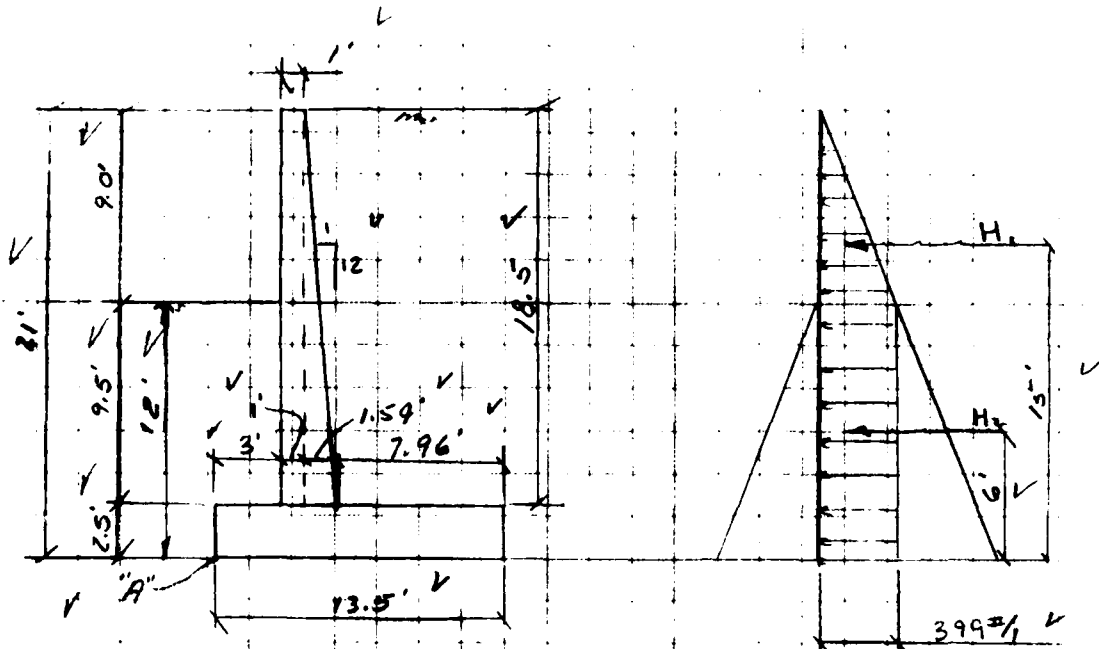
HOWARD NEEDLES TAMMEN & BERENSON

# HNTB

CALCULATIONS FOR

MADE BY C.D. DATE 7/15/75 JOB NO. 40-4  
 CHECKED BY J.A.J. DATE 4/21/75 SEC. NO. 5-43  
 SHEET NO. 5-43

CUT OFF, 1.74 ACN, 1.2.



FOR CALC - EARTH PRESSURE

$$\begin{aligned} \gamma_s &= 133 \text{ lb/ft}^3 \\ \gamma_{\text{sat}} &= 120.3 \text{ lb/ft}^3 \end{aligned}$$

SEE SN 11  
 H/T 15

$$H_1 = 399 \times 9.0 \div 2 = 1,795.5 \text{ ft} \times 15' = 26,925 \text{ ft}^2$$

$$H_2 = 399 \times 12 = 4,788 \text{ ft} \times 6' = 28,728 \text{ ft}^2$$

$$\Sigma H = 6,583 \text{ ft} \quad \text{OTM} = 55,653 \text{ ft}^2$$

HOWARD NEEDLES TAMMEN & BERENDSON CONSULTING ENGINEERS

# HNTB

CALCULATIONS FOR

COY. SICK, J. TAMMEN

MADE BY

DATE

JOB NO.

CHECKED BY

DATE

SEC. NO.

JOB NO.

SEC. NO.

SHEET NO.

5-44

CONSULTING ENGINEERS

HOWARD NEEDLES TAMMEN & BERGENDOFF

$$\begin{array}{lcl}
 \text{CONC} & \text{FTG} & .150 \times 2.5 \times 13.5 = 5.06 \times 6.75 = 34.16 \\
 & \text{STEM} & .150 \times 1.0 \times 18.5 = 2.78 \times 3.5 = 9.73 \\
 & \text{STEM} & .150 \times 18.5 \times 1.54/2 = 2.14 \times 4.51 = 9.7 \\
 \Sigma \text{ CONC} & & 9.98 \times 53.6
 \end{array}$$

$$\begin{array}{lcl}
 \text{EARTH} & \text{TOE} & .133 \times 2.0 \times 9.5 = 2.79 \times 1.5 = 5.7 \\
 & \text{BANK} & .133 \times 18.5 \times 1.54/2 = 1.93 \times 5.03 = 9.5 \\
 & \text{HEEL} & .133 \times 18.5 \times 7.96 = 19.59 \times 9.52 = 186.5 \\
 \Sigma \text{ CONC} + \Sigma \text{ EARTH} & & 35.25 \times 255.3
 \end{array}$$

$$R.M. \div O.T.M. = 255.3 \div 55.7 = 4.58 > 2.0$$

$$\Sigma H \div \Sigma U = 6.58 \div 35.25 = 0.187 < 0.33$$

$$\Sigma M_{\text{O.T.}} \div P \cdot A = 255.3 - 55.7 = 199.6$$

$$e = 199.6 \div 35.25 = 5.66 \text{ ft}$$

$$e = 6.75 - 5.66 = 1.09 \text{ ft}$$

$$S.M. = 6h^2 \div 6 = 1 \times 13.5 \div 6 = 30.4 \text{ ft}^2$$

$$\text{pressure} = \frac{V}{A} \pm \frac{Ve}{S.M.} = \frac{35.25}{13.5} \pm \frac{35.25 \times 1.09}{30.4}$$

$$= 2.61 \pm 1.26 = 3.87 \text{ ft}^2 \div 1.35 \text{ ft}^2$$

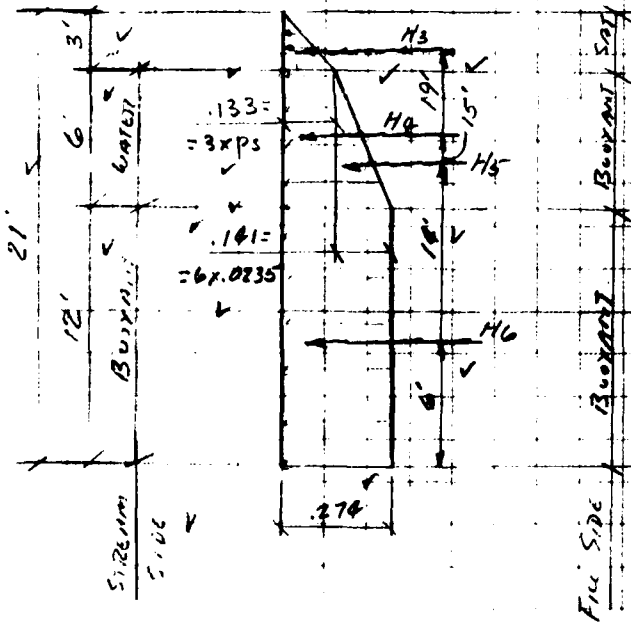
$$0.6 \text{ ft}^2 \div 1.35 \text{ ft}^2$$



# HNTB

MADE BY L.D. DATE 4/2/75 JOB NO. 464  
 CHECKED BY J.K.T. DATE 4.21.75 SEC. NO.   
 SHEET NO. 5-45

CALCULATIONS FOR Check Buoyant



SATURATION\*

$$m_s = .133 \text{ } \checkmark$$

$$p_s = .0443 \text{ } \checkmark$$

Buoyant\*

$$B = .0704 + .0624 = .133 \text{ } \checkmark$$

$$p_B = .0235 + .0624 = .0859 \text{ } \checkmark$$

\* See SH 111 A-1 (NINTAR)

H3	$.133 \times 3 \div 2$	$= .20 \text{ } \checkmark$	$\times 19' \text{ } \checkmark$	$= 3.8 \text{ } \checkmark$
H4	$.133 \times 6$	$= .80 \text{ } \checkmark$	$\times 15' \text{ } \checkmark$	$= 12.0 \text{ } \checkmark$
H5	$.141 \times 6 \div 2$	$= .42 \text{ } \checkmark$	$\times 14' \text{ } \checkmark$	$= 5.9 \text{ } \checkmark$
H6	$.274 \times 12$	$= 3.29 \text{ } \checkmark$	$\times 6' \text{ } \checkmark$	$= 19.7 \text{ } \checkmark$
ΣH		$= 4.71 \text{ } \checkmark$	DTM = 41.4 \text{ } \checkmark	

FOR R.M. ADD 6' OF WATER ON TOP TO THE RESULT OF SATURATION ENDS (NINTAR)

Σ Area x Ch	$35.25 \text{ } \checkmark$	$\times 1.5 = 52.8 \text{ } \checkmark$
6' water $3 \times 6 \times .0624$	$= 1.12 \text{ } \checkmark$	$\times 1.5 = 1.7 \text{ } \checkmark$
Buoy $18 \times .0524 \times 13.5$	$= -15.19 \text{ } \checkmark$	$\times 6.75 = -102.5 \text{ } \checkmark$
	$21.18 \text{ } \checkmark$	$154.5 \text{ } \checkmark$

$$R.M. - DTM = 154.5 \div 41.4 = 3.73 > 2.0 \text{ } \checkmark$$

$$\Sigma H \div \Sigma U = 4.71 \div 21.18 = .222 < .333 \text{ } \checkmark$$

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HOWARD NEEDLES TAMMEN & BERGENDOFF CONSULTING ENGINEERS

# HNTB

CALCULATIONS FOR

MADE BY C.D. DATE 9/10/71 JOB NO. 82  
 CHECKED BY J.K.J. DATE 4/21/75 SEC. NO. 5-46  
 SHEET NO. 5-46

$$EM \text{ at } A'' = 154.5 - 41.4 = 113.1'$$

$$Rc = 113.1 \div 21.18 = 5.34' \text{ FROM P. A.}$$

$$E = 6.75 - 5.34 + 1.41'$$

$$P_{\text{pressure}} = \frac{U}{P} + \frac{U}{S.M.} + \frac{21.18}{13.5} + \frac{21.18 \times 1.41}{30.4} =$$

$$= 1.57' + 0.98' = 2.55' + 0.59' = 3.14'$$

O.K. CTS. TYPED 4" H

## STEM STUCCO

Stem Stucco from 400.

$$(S.M. 93) \text{ Stem } \text{Stucco} = 1.80 \times 12.5' + .395 \times 9.5' = 24.05'$$

$$U = 13.15' = 1.8' + .395 \times 9.5' = 5.55'$$

$$170m \text{ at } 25' \text{ up} = 1.80 \times 3 = 5.4'$$

$$U \text{ 9.5' up} = 1.8' K/L$$

Stem Stucco 25' up

$$Q \text{ at } 25' = 2.50 \times 12 - 2 - 2 = 28.00' \text{ Cts. 28"}$$

$$U = (12 + 9) \times 11' = 207' = 16.5' \text{ Cts. 16.5"}$$

$$A \text{ at } 25' = 40.5 \times 12 \div 24 \times .591 = 16.05' \text{ Cts. 16.05"}$$

$$A \text{ at } 170m \text{ up} = 5.4 \times 12 \div 24 \times .591 = 0.22' \text{ Cts. 22"}$$

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HOWARD NEEDLES TAMMEN & BERGENDOFF CONSULTING ENGINEERS

### CALCULATIONS FOR

**MADE BY**

DATE \_\_\_\_\_

JOB NO

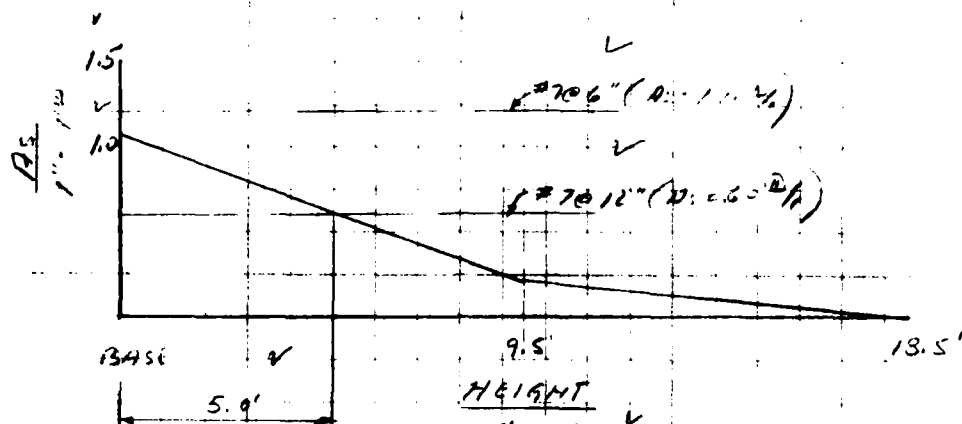
**CHECKED BY**

DATE \_\_\_\_\_

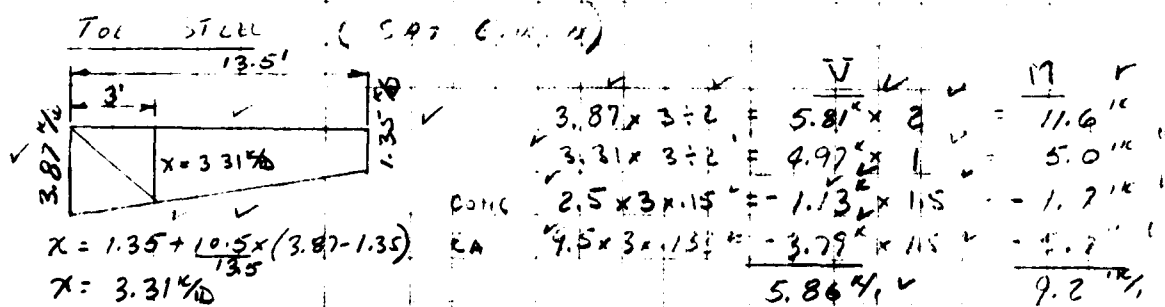
SEC. NO.

SHEET NO.

5-47



$$\text{dev. theor. cut-off} = 12'' + 13.5'' - 4.5'' = 21'' = 1.75'$$
 For spacing of diaphragms use 1.7' (see Spacing)



$$d = 30 - 4 - 1 = 25.5''$$

Dec. 9. 20 ✓ 10. 20 ✓ 11. 20 ✓ 12. 20 ✓ 13. 20 ✓ 14. 20 ✓ 15. 20 ✓

Box 120 12" (4.6 3/4) 120 5700  
1110 1294

# HNTB

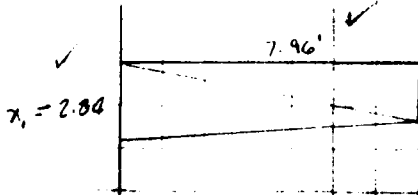
CALCULATIONS FOR

MADE BY \_\_\_\_\_ DATE \_\_\_\_\_ JOB NO. \_\_\_\_\_  
 CHECKED BY 2.1.7 DATE 4/22/75 SEC. NO. \_\_\_\_\_  
 SHEET NO. 5-48

HEAVY TRUCK (27 CARTS)

$$X_1 = 1.35 + \frac{7.96}{13.5} (3.81 - 1.35)$$

$$X_1 = 2.84$$



$$\begin{aligned} 1.35 \times 7.96 \div 2 &= 5.31' \times 5.31' = 28.5' \\ 2.84 \times 7.96 \div 2 &= 11.30' \times 2.65' = 30.0' \\ 1.35 \times 2.5 \times 7.96 \div 18 &= 2.99' \times 3.98' = 11.9' \\ \text{C.A. } 18.5 \times 7.96 \div 18 &= 19.59' \times 3.53' = 78.0' \\ &= 5.91' \times 31.9' \end{aligned}$$

$$A_s = 31.2 \times 12 \div 70 \times .891 \times 15.2 = 0.83 \text{ in}^2$$

USE #6 @ 6" (A\_s = 0.88 in^2)

TEMP & SHrinkAGE CORR.

REF. CM 1110-2-2103

Full length use approx. 12' x 12' x 12' @ 12' x 12' x 12'

$$A_{s/corr} = A_s / f_{cor} = \frac{0.09}{2} \times (1.0 + 2.54) \times \frac{12}{2} \times 12 = .51 \text{ in}^2$$

(see 5463) RESTRICTIONS, CASE 2, 18' - 2.5' = 15.5'

$$\therefore 15.5 \div 4 = 3.9' \text{ say } 3'$$

For lower 3' use #7 @ 12" (1.12 in^2)

For remainder of wall use #6 @ 6"

$$A_{s/corr} = \frac{0.025}{2} (1.0 + 2.54) \times \frac{12}{2} \times 12 = .32 \text{ in}^2$$

For remainder of wall use #6 @ 12" (A\_s = .31 in^2)

USE #6 @ 12" FOR WALL (A\_s = .31 in^2)

HOWARD NEEDLES TAMMEN & BERGENDOFF CONSULTING ENGINEERS

# HNTB

### CALCULATIONS FOR

MADE BY

DATE \_\_\_\_\_

**JOB NO.**

**CHECKED BY**

DATE \_\_\_\_\_

SEC. NO.

SHEET NO.

1907 Bonanzas of Chile & Peru  
(see Notes Bonanzas)

Av. Vol. Corc.  $C_{ms} \cong (12 + 18.25) \times \frac{1}{2} + 6.25 \times \frac{1}{2} \times 5 = 39.0 \text{ ft.}^3$

$$V = 15 \times 39.4 = 591^{\text{m}}$$

$M = 5.91 \times 10^{21} \text{ g}$

$$u = 2.5 \times 10^{-4} - 4 + 2 = 555$$

A.  $19.8 \times 10 \div 20 \times 991 \times 55 \div 12 = 2.2$   
2.  $5 \times 50000 \div 15 = 1666.67$

(Co. Ser. No. 22)  $\sigma_d = .55 \times 2 \times (f_c)^{1/2} = 60 \text{ psi}$

$$D_L = 5,910 \div (39.25) \text{ L} \times 6.25 \text{ mL} = 5.2 \text{ mL}$$

Shoreline map of N. 21

6-11-58 12

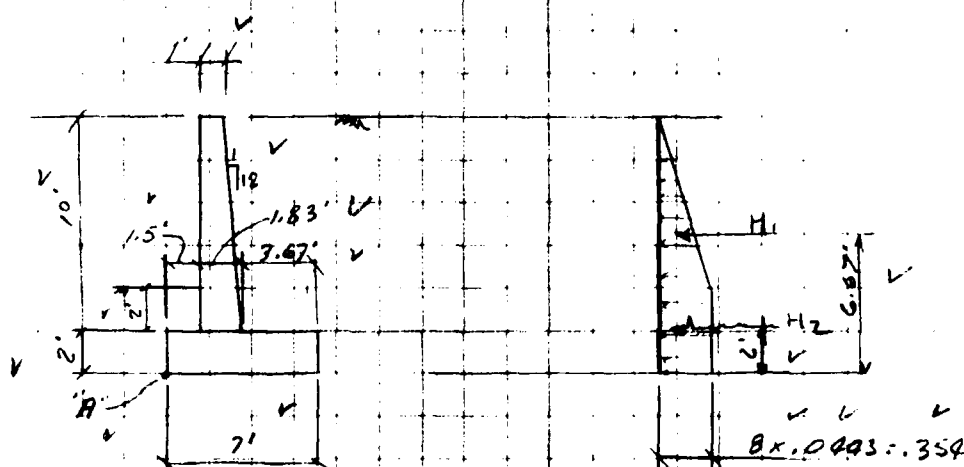
**LEONARD NEEDLES TAMMEN & BERGENDOFF**  
**CONSULTING ENGINEERS**

# HNTB

CALCULATIONS FOR GRILLAGE DESIGN

MADE BY JKT DATE 8/16/75 JOB NO. 412-1  
 CHECKED BY JKT DATE 4/26/75 SEC. NO. 5-50  
 SHEET NO. 5-50

Retaining Wall



$$W_s = 133 \text{ lb/ft} \quad \text{SH 112 A 1}$$

$$P_s = 89.5 \text{ lb/ft} \quad \text{HNT 4 B}$$

$$H_1 = 0.354 \times 3 = 1.062 \text{ k}$$

$$H_2 = 0.354 \times 4 = 1.416 \text{ k}$$

$$2.84 \text{ k}$$

$$12.2 \text{ k}$$

7.9 k6.0 k12.2 k

CONC	1.50 x 7 x 2	=	2.10 k	x	3.5'	=	7.4 k
STEEL	1.50 x 1 x 12	=	1.50 k	x	2.3'	=	3.5 k
STEEL	1.50 x 10 x 3.6	=	0.62 k	x	2.79'	=	1.7 k
Σ CONC			4.22 k				12.1 k

CONC	7.5	133 x 2 x 1.5	=	0.40 k	x	0.75'	=	0.3 k
STEEL	1.50 x 10 x 8.3/2	=	0.55 k	x	3.05'	=	1.7 k	
STEEL	1.50 x 10 x 3.67	=	0.88 k	x	5.17'	=	4.6 k	

Σ CONC + STEEL = 10.05 k

RM = 17.4 x 23 = 400.2

17.4 x 23 = 400.2

HOWARD NEEDLES TAMMEN & BERGENDOFF CONSULTING ENGINEERS

# HNTB

CALCULATIONS FOR

MADE BY

C.D.

DATE

8/16/75

JOB NO.

9204

CHECKED BY

J.K.T.

DATE

8/22/75

SEC. NO.

SHEET NO.

5-51

Cor. Green, Tammen, I.

$$E.M.C.P. "A" \quad 39.3 - 12.2 = 27.1'$$

$$Result is \quad 27.1 - 10.05 = 17.05' \text{ From Pt. "A"}$$

$$C = 3.5 - 2.7 = 0.8'$$

$$S.M. \quad 1/2 \times 12 \div 6 = 1 \times 72 \div 6 = 8.17 \text{ ft}^2$$

$$pressure = \frac{10.05}{7} + \frac{.8 \times 10.05}{8.17} = 1.44 + .98$$

$$= 2.42 \text{ ft}^2$$

$$= 0.46 \text{ ft}^2$$

$$OR \quad C + 1/10$$

BOYD'S CONDITION WILL NOT GOVERN.  
(SEE CALC. FOR 12.5' HIGH STEM)

STEM STILL

SAP. COND. WILL GOV.

$$M_{max} @ Base = 1.42 \times 4.67 + .354 \times 2 \times 1 = 7.3 \text{ ft}^2$$

$$U @ Base = 1.42 + .354 \times 2 = 2.13 \text{ ft}^2$$

$$M_{max} @ 2' = 1.42 \times 2.67 = 3.8 \text{ ft}^2$$

$$U @ 2' = 1.42 \text{ ft}^2$$

$$J @ Base = 1.83 \times 12 - 8 - 2 = 17.86 \text{ ft}^2 \quad \text{At } 17.5'$$

$$A = 7.2 \times 12 \div 20 \times .891 \times 17.5 = .28 \text{ ft}^2$$

$$U @ 2' @ 12' Full Height At .31 \text{ ft}^2$$

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HOWARD NEEDLES TAMMEN &amp; BERGENDOFF CONSULTING ENGINEERS

# HNTB

CALCULATIONS FOR

Box Gully

MADE BY

DATE

JOB NO.

CHECKED BY

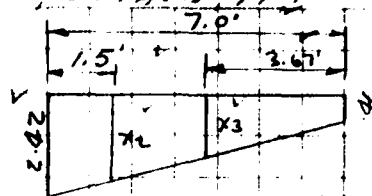
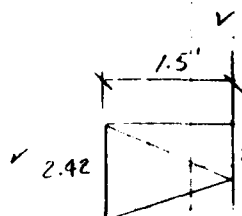
DATE

SEC. NO.

SHEET NO.

5-22

Top Steel



$$X_2 = .46 + \frac{5.5}{7} (2.42 - .46)$$

$$X_2 = 2.00$$

$$2.42 \times 1.5 \div 2 = 1.82$$

$$2.00 \times 1.5 \div 2 = 1.50$$

$$CIRC. .15 \times 2 \times 1.5 = 0.45$$

$$CIRC. .133 \times 2 \times 1.5 = 0.40$$

V	M
1.82	1.0 = 1.8
1.50	1.5 = 0.84
0.45	.75 = 0.31
0.40	.75 = 0.31
2.42	2.0

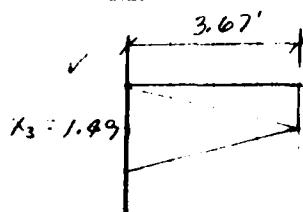
$$A_1 = 2.4 \times 12 \div 70 \times 885 \times 19.5 = 19.5$$

$$A_2 = 2 \times 12 \div 70 \times 885 \times 19.5 = 19.5$$

$$0.07 \frac{1}{11}$$

USE #5 @ 12" (REIN. MAIN STEEL)

HEEL STEEL



SEE SKETCH ABOVE

$$X_3 = .46 + \frac{3.17}{7} (2.42 - .46)$$

$$X_3 = 1.49$$

$$.46 \times 3.67 \div 2 = 0.84$$

$$1.49 \times 3.67 \div 2 = 2.73$$

$$.15 \times 3.67 \times 2 = 1.10$$

$$.133 \times 3.67 \times 2 = 0.98$$

V	M
0.84	2.45 = 2.1
2.73	1.22 = 3.3
1.10	1.54 = 2.0
0.98	1.84 = 2.0
2.42	5.6

$$A_2 = 5.6 \times 12 \div 70 \times 885 \times 19.5 = 19.5$$

USE #5 @ 12" (REIN. MAIN STEEL)

HOWARD NEEDLES TAMMEN & BERGENDOFF CONSULTING ENGINEERS



# HNTB

CALCULATIONS FOR

Box Culvert, Ithaca, N.Y.

MADE BY L.D. DATE 4/17/75 JOB NO. 4607  
 CHECKED BY 2KJ DATE 4/22/75 SEC. NO. 5-53  
 SHEET NO. 5-53

TEMP. + SMITHING STEEL ✓

REF. EMT 1112-2-2103 PG 5 b(3)

MENT. REINFORCED ③ DIVE 6260

$$A_{s/FACE} = \frac{.004}{2} \times (1 + 1.83) \times \frac{12}{2} \times 12 = .41 \%$$

USE #6 @ 12" (A<sub>s</sub> = .44%) ✓

SE 62 } REINFORCED EDGE TO 18' - 2.5' = 15.5'

$$15.5' \div 4 = 3.6'$$

FOR LENGTH 3' USE #6 @ 12" ✓

FOR REMAINDER OF HORIZ STEEL PG 5 b(1)

$$A_{s/FACE} = \frac{.0025}{2} (1 + 1.83) \times \frac{12}{2} \times 12 = .25 \%$$

USE #5 @ 12" (A<sub>s</sub> = .314%) ✓

CONSULTING ENGINEERS

HOWARD NEEDLES TAMMEN & BERGENDOFF

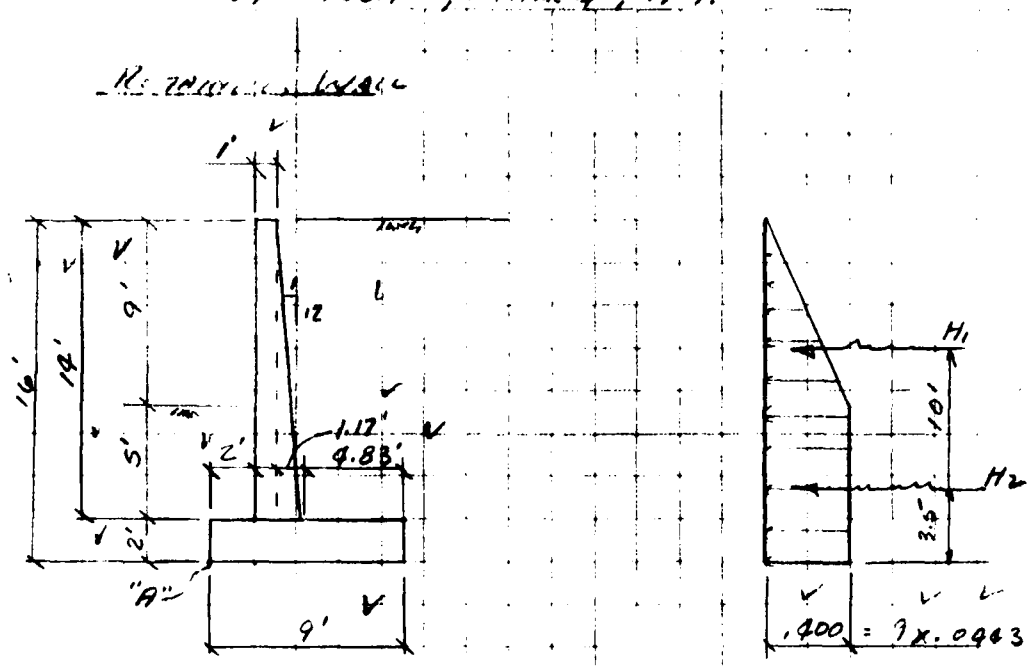
# HNTB

CALCULATIONS FOR

Cox Green, Jamaica, N.Y.

MADE BY C.D. DATE 7/12/72 JOB NO. 9104  
 CHECKED BY Z.H.J. DATE 4.22.75 SEC. NO.         
 SHEET NO. 5-34

HOWARD NEEDLES TAMMEN & BERGENDOFF CONSULTING ENGINEERS



As shown from calc. of H<sub>1</sub> & H<sub>2</sub>, the governing condition is for 13% saturated soil.

133 #/ft<sup>3</sup> } See SM A /  
 90.3 #/ft<sup>3</sup> } H.N.T.C.B.

$$\begin{aligned}
 H_1 &= .4 \times 9 \div 2 = 1.8^{\text{K}} \\
 H_2 &= .4 \times 7 = 2.8^{\text{K}} \\
 \Sigma H &= 4.6^{\text{K}}
 \end{aligned}
 \quad
 \begin{aligned}
 &\times 10 = 18.0^{\text{K}} \\
 &\times 3.5 = 9.8^{\text{K}} \\
 \text{O.T.M.} &= 27.8^{\text{K}}
 \end{aligned}$$

# HNTB

CALCULATIONS FOR

Cox 62071, ITHACA, N.Y.

MADE BY LD DATE 4/12/75 JOB NO. 9209  
 CHECKED BY JKJ DATE 4/22/75 SEC. NO.   
 SHEET NO. 5-55

TAKE MATH. ABOUT PT "A"

		V	V	V	V	M
CONC.	FTG	.150	2x9	F	2.70' x 4.5'	12.2'
	STEM	.150	1x14	F	2.10' x 2.5'	5.3'
	STEM	.150	18x1.17	F	1.23' x 3.39'	4.2'
Σ CONC.					6.03' x 2	21.7'
CAULDRON	TOP	.133	2x5	F	1.33' x 1.0'	1.3'
	BACK	.133	18x1.17	F	1.03' x 3.78'	4.1'
	NOZ	.133	10x4.93	F	8.99' x 6.59'	59.3'
Σ CONC. + CAULDRON					17.44' x 2	86.4'

$$R.I. \div 0.7.M = 86.4 \div 27.8 = 3.11 > 2.0$$

$$\Sigma H \div \Sigma U = 4.6 \div 17.00 = .264 < .333$$

$$\Sigma M \text{ about PT "A"} = 86.4 - 27.8 = 58.6'$$

$$RESULT IS 58.6 \div 17.00 = 3.36 \text{ FROM PT "A"}$$

$$e = 4.5 - 3.36 = 1.14' \quad \Sigma M = 66' \div 6 = 9' \div 6 = 13.5'$$

$$\text{pressure} = \frac{U}{A} \pm \frac{Ue}{S.A.} = \frac{17.44}{9} \pm \frac{17.44 \times 1.14}{13.5}$$

$$= 1.94 \pm 1.47 = 3.26'$$

$$0.47' \text{ max}$$

OK L.C.S. MAX 4' max

HOWARD NEEDLES TAMMEN & BERGENCOFF CONSULTING ENGINEERS

CALCULATIONS FOR Cox Gully, Ithaca, N.Y.

## STEEL

$$MIN \text{ BASE} = 1.8 + 3 + .4 \times 5 \times 2.5 = 19.4''$$

$$DC \text{ BASE} = 1.8 + .0 + 5 = 3.5'$$

$$MIN \text{ S'OP} = 1.8 + 3 = 5.4'$$

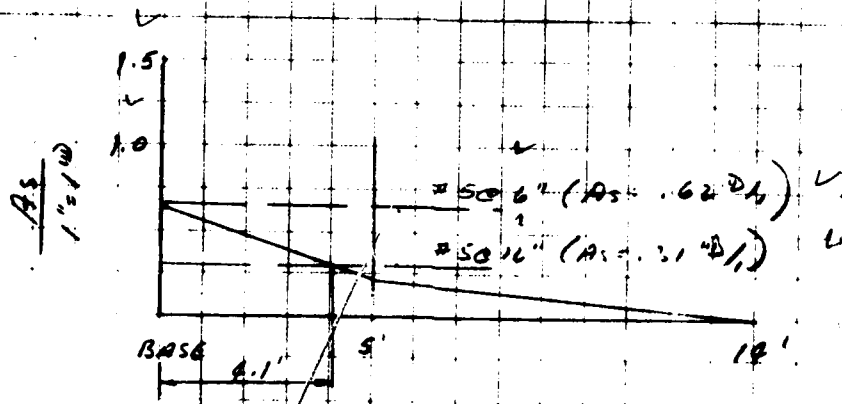
$$US'OP = 1.8'$$

$$dc \text{ BASE} = 26 - 4 - 6 = 21.5''$$

$$d \text{ S'OP} = (12 + 9) - 4 - 6 = 16.5'$$

$$As \text{ BASE} = 19.4 \times 12 \div 20 \times .891 \div 21.5 = 0.61 \text{ D/I}$$

$$As \text{ S'OP} = 5.4 \times 12 \div 20 \times .891 \div 16.5 = 0.22 \text{ D/I}$$



HEIGHT  
 1" = 2'

$$\text{CUT-OFF OF LONG DRAIN} = 4.1 + d = 4.1 + 16.5 = 5.5'$$

USE 12 SINCE  $1.8 \times 10.0 < 12'$

$$\text{SPACE LENGTH} = 1.3 \times 12 = 15.6' \text{ USE } 16'$$

$$\text{REF A.C.I. - 318-71 Pg 44 12.5}$$

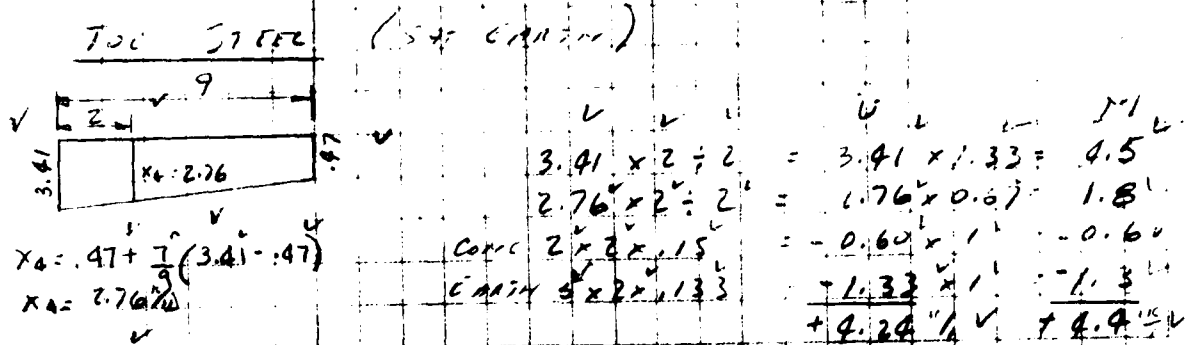
# HNTB

CALCULATIONS FOR

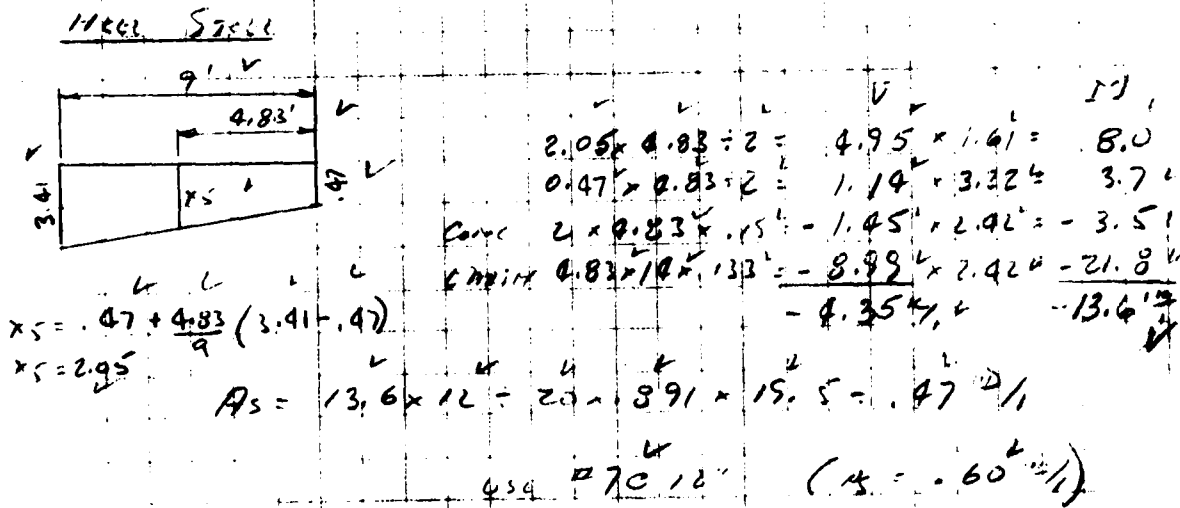
Cox Creek, Indiana, 11/4

MADE BY C.D. DATE 9/13/75 JOB NO. 4104  
 CHECKED BY J & J DATE 4.22.75 SEC. NO.   
 SHEET NO. 5-57

HOWARD NEEDLES TAMMEN & BERENSON CONSULTING ENGINEERS



$d = 24 - 9 - 2 = 13$   
 $A_s = 4.4 \times 12 = 20 \times .891 \times 13 = 15.5$   
 Use #5 @ 12" (Bottom reinforcement)  
 $(A_s = 31 \frac{1}{2})$



# HNTB

MADE BY C.D. DATE 7/14/75 JOB NO. 5104  
CHECKED BY 7AJ DATE 4/23/75 SEC. NO.         
SHEET NO. 5-58

CALCULATIONS FOR Cox Gully, JENNIFER, N.Y.

## TEMP & SHRINKAGE REINF

REF. E.11 1110-2-2103 ✓  $R_s$  5 # (3) ✓

Mem. REINFORCED @ ONE CIRC ✓

$$P_s / f_{mc} = \frac{.004}{2} (1 + 2.12) \times \frac{12}{2} \times 12 = .26 \text{ in.}^2/\text{ft.}$$

USE # 7 @ 12" (AS = .60 in.²/ft.) For leave  
3' OF GULLY ✓

FOR REMAINING HORIZ STRIP  $R_s$  5 # (1)

$$P_s / f_{mc} = \frac{.0025}{2} (1 + 2.12) \times \frac{12}{2} \times 12 = .29 \text{ in.}^2/\text{ft.}$$

USE # 5 @ 12" (AS = .31 in.²/ft.) ✓

HOWARD NEEDLES TAMMEN & BERENDSON CONSULTING ENGINEERS

# HNTB

CALCULATIONS FOR

Cor. Glen, I-190A

MADE BY LD  
CHECKED BY J.A.J

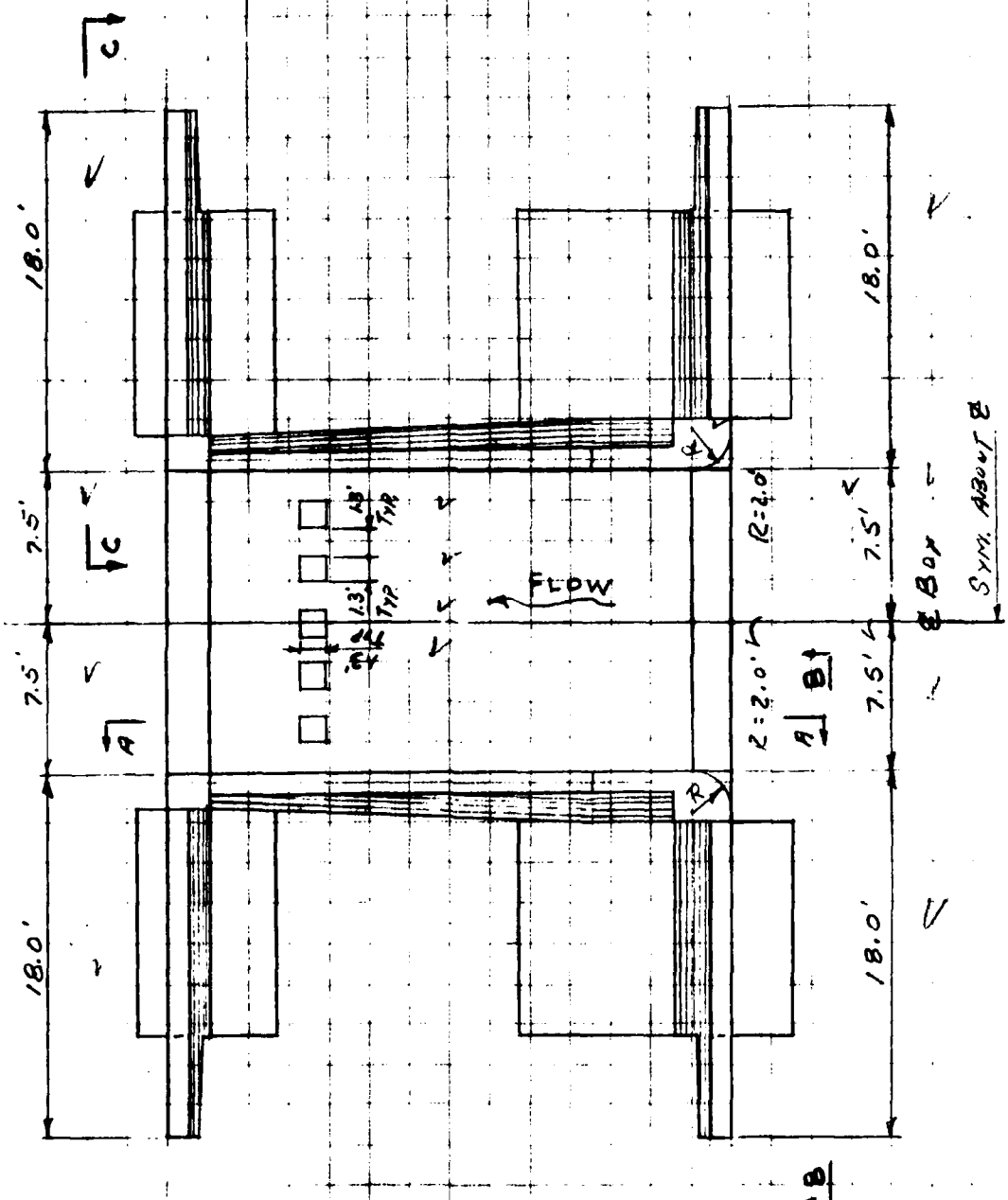
DATE 9/11/75  
DATE 4-23-75

JOB NO. 92-4

SEC. NO. 5-55

SHEET NO. 5-55

HOWARD NEEDLES TAMMEN & BERENDORFF CONSULTING ENGINEERS



SCHEME 1/0.1  
(DRAWN BY 142142)  
1" = 1'-0"

NOTE:  
(1) FOR CONSTRUCTION PROCEEDURE SEE SHEET 594.

# HNTB

CALCULATIONS FOR

COY GLEN, ITHACA, N.Y.

MADE BY L.V. DATE 5/6/75 JOB NO. 0204  
CHECKED BY J.H.J. DATE 4.23.75 SEC. NO.   
SHEET NO. 5-59A

## CONSTRUCTION PROCEDURE SCHEME NO. 1

- 1) EXCAVATE FOR BOX CONSTRUCTION V  
(ELEV 383.3 DROP STR 1/1) V  
( " 379.5 " " 2 ) V
- 2) CONSTRUCT BOX V
- 3) EXCAVATE FOR WALL CONSTRUCTION V  
(DROP STR 1/1 TO BE EXCAVATED TO  
✓ ELEV 375, THEN BACKFILL WITH SUITABLE  
MATERIAL TO ELEV 383.3)  
(DROP STR NO 2 TO BE EXCAVATED TO  
ELEV. 379.5) V  
✓ NOTE: CARE SHALL BE EXERCISED SO AS  
NOT TO UNDERMINE THE NEWLY  
CONSTRUCTED BOX
- 4) CONSTRUCT WALLS V

CONSULTING ENGINEERS

HOWARD NEEDLES TAMMEN & BERENDSON



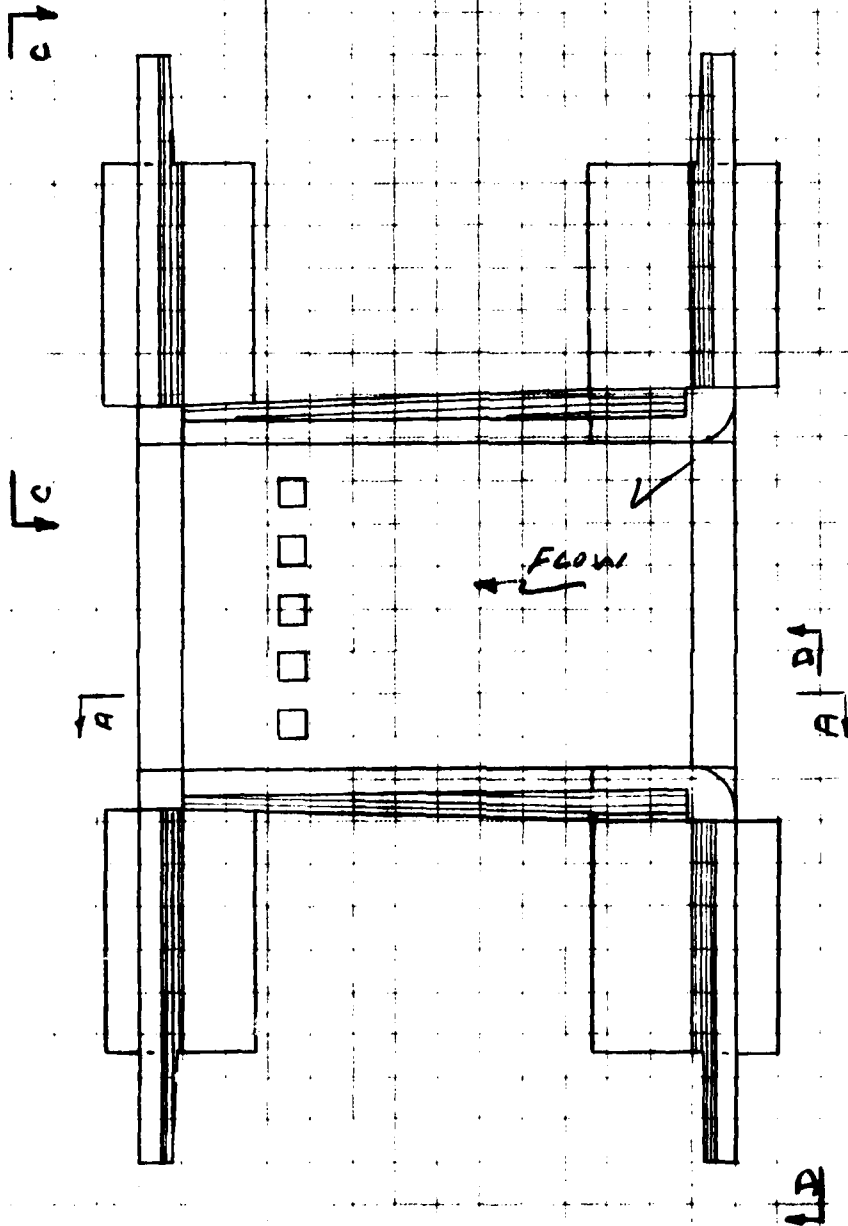
# HNTB

CALCULATIONS FOR

Cox Glen, Ithaca, N.Y.

MADE BY L.D. DATE 9/14/75 JOB NO. 4104  
 CHECKED BY J & J DATE 4/22/76 SEC. NO.   
 SHEET NO. 5-59B

HOWARD NEEDLES TAMMEN & BERENSON OFF CONSULTING ENGINEERS



SCHEME No 2  
 (Drawn by Mr. 142)  
 1/8" = 1'-0"

NOTE:

- (1) For Corrosion & Tension, Proceed with S.H. 59C
- (2) For Dim. for S.H. 59.

# HNTB

CALCULATIONS FOR

COY GLEN, ITHACA, N.Y.

MADE BY C.D. DATE 2/6/75 JOB NO. 4204  
CHECKED BY J.R.T. DATE 4.23.75 SEC. NO. 5-59C  
SHEET NO. 5-59C

## CONSTRUCTION PROCEDURE SCHEME NO 2

- 1) EXCAVATE FOR BOX CONSTRUCTION  
( GLEN 383.3' DROP STR NO 1 )  
( " 379.5' " " " )
- 2) CONSTRUCT BOX
- 3) EXCAVATE FOR WALL REINFORCEMENT.  
( DROP STR NO 1 TO BE EXCAVATED TO  
ELEV 375, THEN BACKFILL UP STREAM  
WALLS TO ELEV 383.3, AND DOWN STREAM  
WALLS TO ELEV. 383.3 WITH SUITABLE  
MATERIAL )  
( DROP STR NO 2 TO BE EXCAVATED TO  
ELEV 379.5, THEN BACKFILL UP STREAM  
WALLS TO ELEV 379.5 WITH SUITABLE  
MATERIAL )  
NOTE: CARE SHALL BE EXERCISED SO  
AS NOT TO UNDERMINE THE  
NEWLY CONSTRUCTED BOX.
- 4) CONSTRUCT EAKS.

HOWARD NEEDLES TAMMEN & BERENDSON CONSULTING ENGINEERS



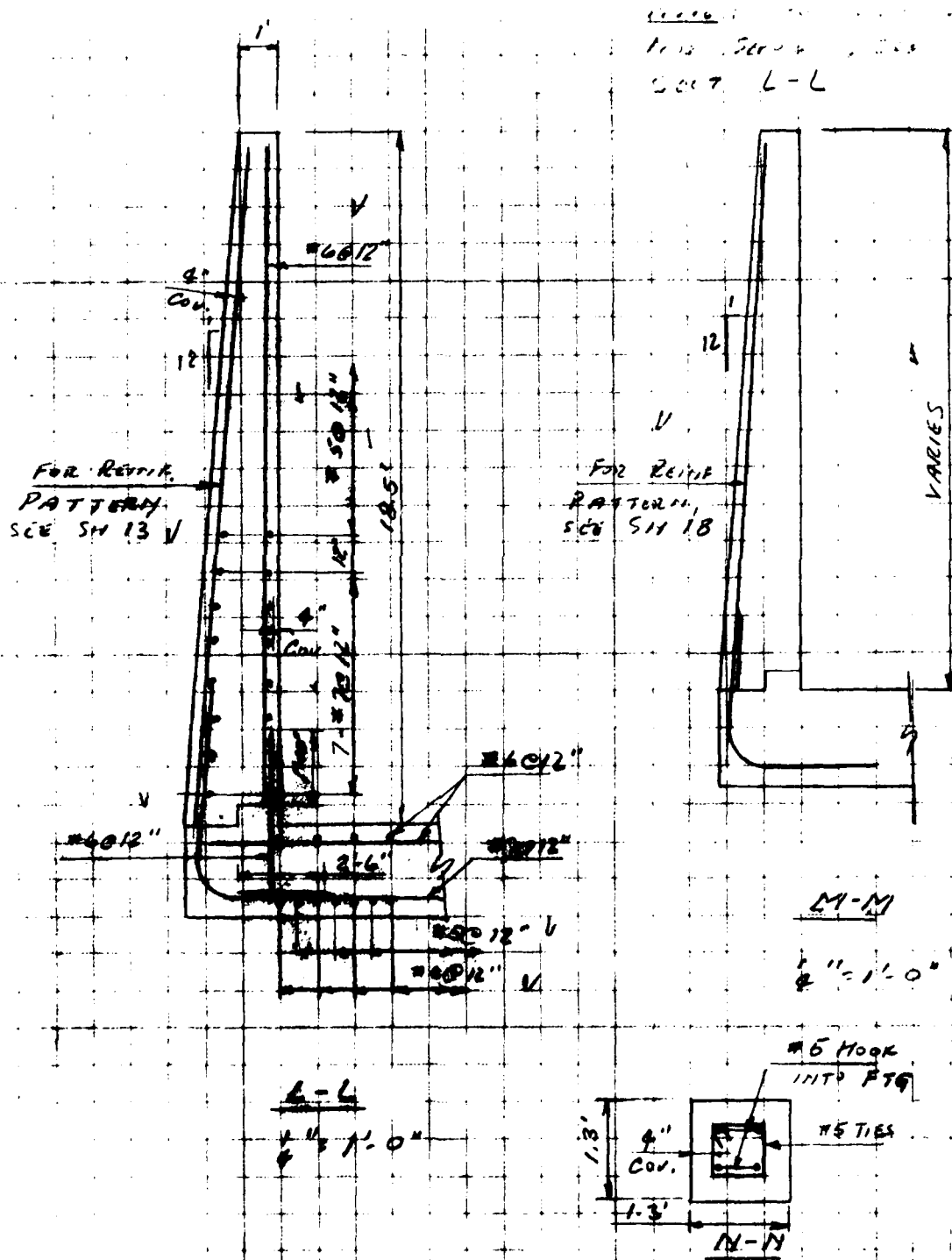
# HNTB

CALCULATIONS FOR

COY 6001, ITHACA, NY

MADE BY L.I. DATE 4/18/25 JOB NO. 4104  
CHECKED BY J.H.G. DATE 4/23/25 SEC. NO. 5-61  
SHEET NO. 5-61

HOMER BIDDLE TAYLOR & ASSOCIATES CONSULTING ENGINEERS



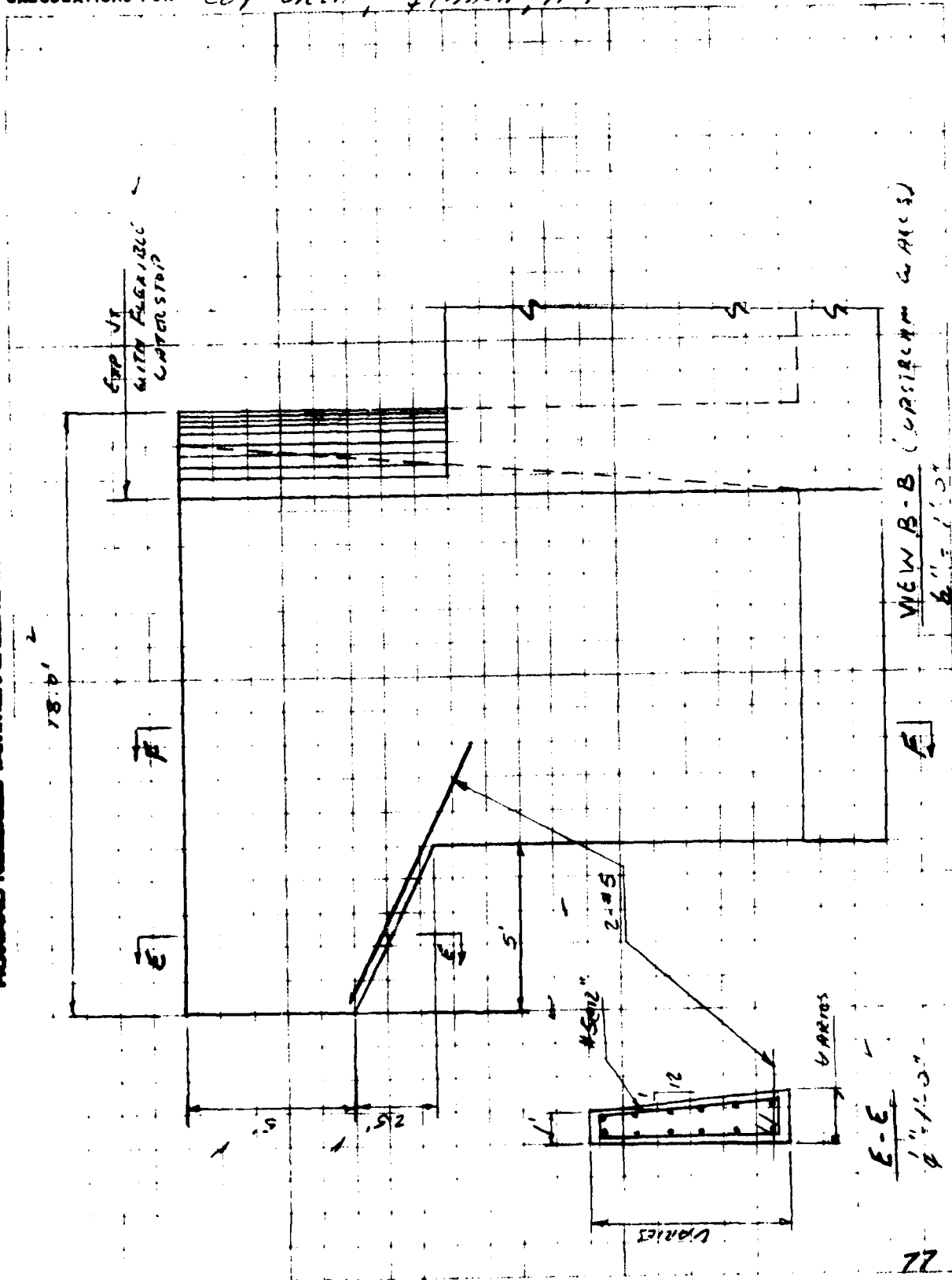
# HNTB

CALCULATIONS FOR

COY GLEN, ILLINOIS, N.Y.

MADE BY L.D. DATE 4/16/77 JOB NO. 4104  
 CHECKED BY J.K.J. DATE 4/25/75 SEC. NO. 5-62  
 SHEET NO. 5-62

HOWARD NEEDLES TAMMEN & BERENSON CONSULTING ENGINEERS



VIEW B-B (CURSIVE WALLS)

E-E 1/4\"/>

# HNTB

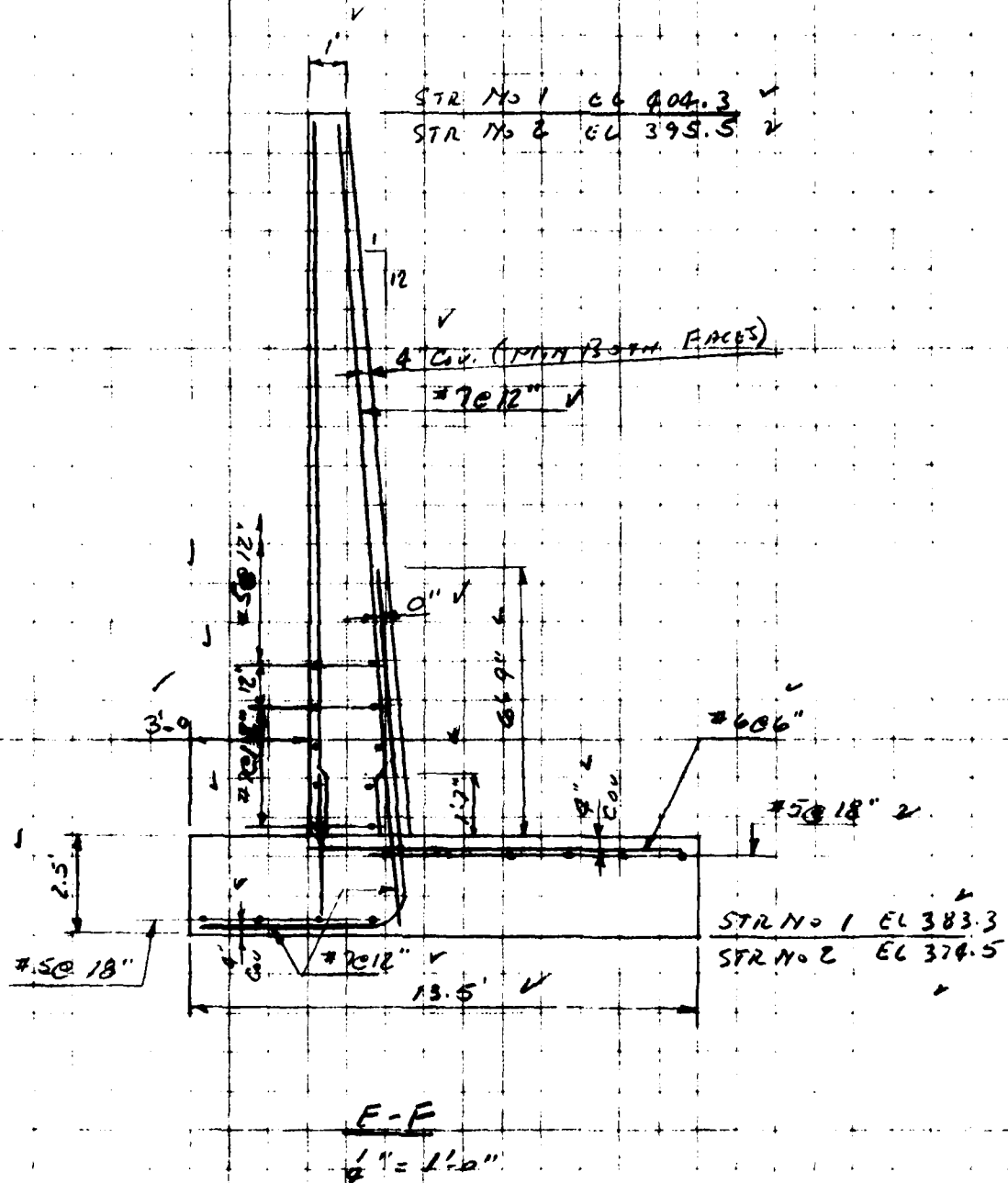
CALCULATIONS FOR

CEY GREEN, 27 INCH, 11 1

MADE BY 1.7 DATE 7/1-72 JOB NO. 9104  
 CHECKED BY 7.87 DATE 7/1-72 SEC. NO. 5-63  
 SHEET NO. 5-63

HOWARD NEEDLES TAMMEN & BERENDSON CONSULTING ENGINEERS

HOWARD NEEDLES TAMMEN & BERENDSON CONSULTING ENGINEERS



### CALCULATIONS FOR

MADE BY CD DATE 9/17/75 JOB NO. 3104  
CHECKED BY JAZ DATE 4.24.75 SEC. NO. \_\_\_\_\_  
SHEET NO. 5-64

COY BLEN, INHACA, N. Y.

## HOWARD NEEDLES TAMMEN & BERGENDORFF

Nov 20 11 45 AM 2011

STRA 1 66395.8  
STRA 2 66386.9

18.01

EXP. OF CITY  
FLEX SATELITE

३

STN N<sup>o</sup> 1 66 383.3  
STN N<sup>o</sup> 2 66 370.5

DATE C-C

DOUGLAS TRENN (c. 1865)  
"4" = 1'-0"

5-9

79

# HNTB

CALCULATIONS FOR

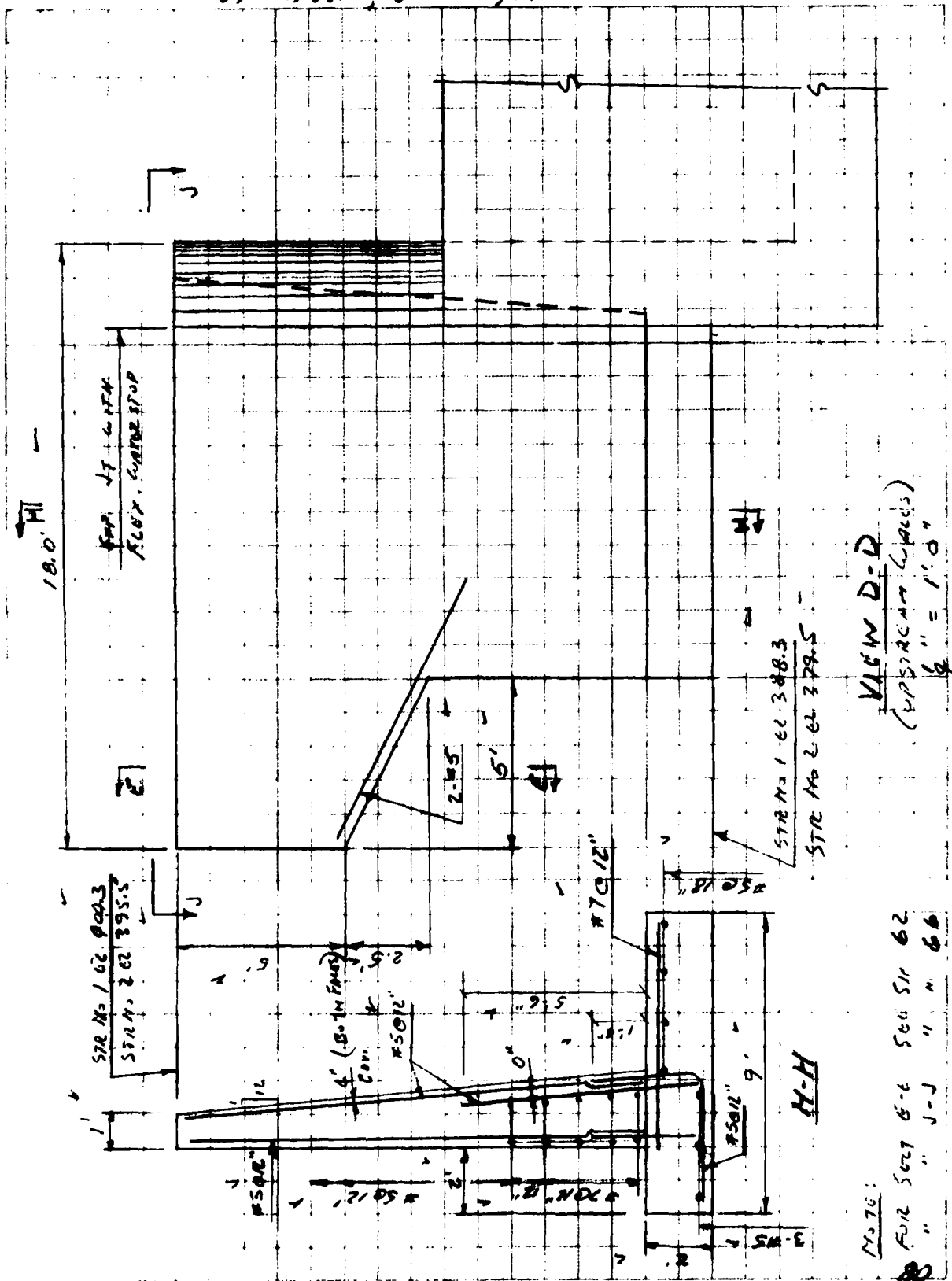
COY GLEN, TAMMEN, NY

MADE BY L.V.  
CHECKED BY T.H.T.

DATE 8/12/75  
DATE 4.20.77

JOB NO. 9204  
SEC. NO. 5-65  
SHEET NO. 5-65

HOWARD NEEDLES TAMMEN & BERENSON CONSULTING ENGINEERS



NOTE:  
FOR SET 6-6 SEE SH 62  
" " J-J " " 66



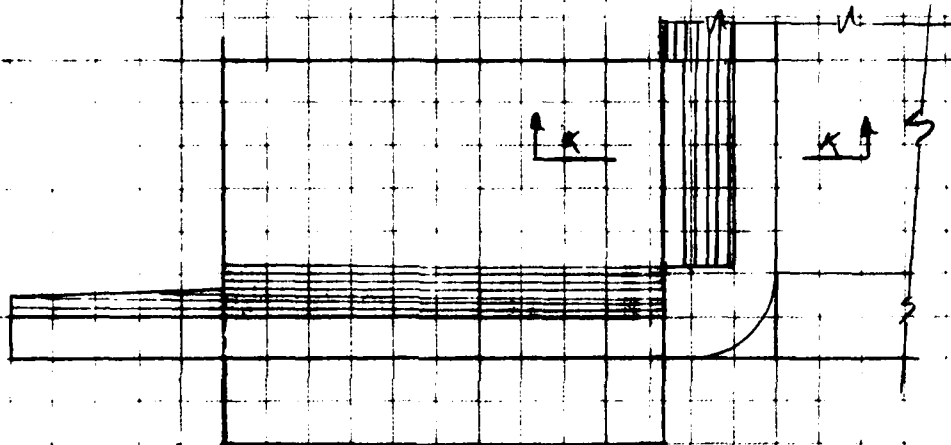
# HNTB

CALCULATIONS FOR

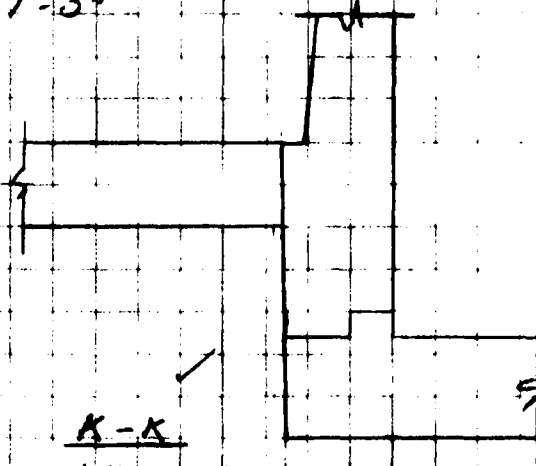
Coy Glen, Ithaca, N.Y.

MADE BY C.D. DATE 2/18/75 JOB NO. 4004  
CHECKED BY T.K.J. DATE 4.24.75 SEC. NO. 5-66  
SHEET NO. 5-66

HOWARD NEEDLES TAMMEN & BERENSON  
CONSULTING ENGINEERS



✓  
VIEW JJ  
(UPSTREAM WALL)  
 $\frac{1}{4}'' = 1'-0''$



✓  
K-K  
 $\frac{1}{4}'' = 1'-0''$

81

## 2. SOILS ANALYSIS

2.1 This section outlines the methods used to determine the lateral pressures for design of the drop structures, the stability of the drop structure excavation, and the design of the sheet pile alternate for the drop structure wingwalls.

2.2 For the design of the drop structures, at-rest earth pressures were used while for the design of the wingwalls, active pressures were used, Sheet A-1. The backfill for which these pressures were calculated was a sand having a natural unit weight of 120 pcf, a saturated unit weight of 133 pcf and an angle of internal friction of 30 degrees.

2.3 The soil profile for these designs is based on borings made for the Cayuga Inlet Flood Protection construction and is shown on Sheet A-2.

2.4 The shear strength of the clay above El. 375 is important to the design of the excavations for the two drop structures and the sheet pile wingwall alternates. From spoon penetration resistances, average N-2.5, it was initially estimated that the cohesion of the clay was 312 psf, Sheet A-5. Subsequently a limited number of torevane and pocket penetrometer tests were made on the clay at the site, Sheet A-7, from which it was concluded that the value of the clay cohesion was 600 psf or greater.

2.5 The stability of the excavations for the cohesion values of 312 psf and 600 psf were checked, Sheets A-5 and A-5a. For a cohesion value of 600 psf and a safety factor of 1.5, a 0.5H:1V slope can be used

for Structure No. 1, and a 1H:1V slope can be used for Structure No. 2. However, because of potential seepage problems at the excavation bottom for Structure No. 2 and potential bottom heave at Structure No. 1, it is recommended that dewatering of the sand and gravel layer below El. 375 be used at both structures.

2.6 Structure No. 2 will be founded on the dense sand and gravel stratum below El. 375 and therefore should have good bearing. The same condition applied for the concrete cantilever wingwall alternates for this structure which will be founded at the same level. Structure No. 1 will be founded on clay at El. 383. The normal dead load bearing capacity safety factor for the box structure is 4.0 and the minimum safety factor for dead plus maximum live load is 2.9. For the concrete cantilever wingwall alternate, the minimum safety factor ranges from 1.6 for the downstream wall to 1.0 for the upstream wall. Should it be desired to utilize the concrete cantilever wingwall alternate of this structure, it is recommended that the clay beneath the wall be removed down to the sand and gravel stratum at El. 375 and backfilled with sand or gravel to the footing elevation. Since the backfill will not be compacted, a maximum wingwall footing bearing value of 2 tsf should be used so to limit settlement of the wingwall. Bearing capacity calculations for a cohesion of 600 psf are given on Sheet A-7.

2.7 Cantilever steel sheet pile wall alternates were designed for both drop structures. Active earth pressures were utilized for these designs.

Above the channel bottom a granular backfill was assumed with full water pressure for the design channel depth of five feet. Below the channel bottom clay soil was assumed and the differential water level was assumed to decrease linearly to zero at El. 375 since the sand and gravel layer below this level is expected to balance the water pressures in each side of the sheeting. The design for the cantilever steel sheet pile wingwall alternates for both structures is included in Sheets A-8 to 31. Design summaries are given on Sheet A-32.

## Soil Pressures for Structure Design.

Sta. No.	E.I. @ top	E.I. @ bottom (Ref. Inc. D-2)
1	401	384 ±
2	393	375 ±

Soil Conditions (Ref borings SS-D & SS-1) (See Sh. A-2)

E.I. 401 to 384 CL, N=1-4 2.5 av

Est. NWC = 30% (Ref. Boring SS-D)

For NWC=30%,  $G_s = 2.67$   $\gamma_{dry} = 93 \text{ pcf}$  (Sh. A-3)

$$\gamma_{sat} = 1.30 \times 93 = 121 \text{ pcf}$$

$$\gamma_{buoyant} = 121 - 62.4 = 58.6 \text{ pcf}$$

E.I. 393 to 375 375

E.I. 393 to 383 CL as above

below 375 383 to 375 Sand, Gravel GP GM N=11 to 38, av=23

$D_r = 40\%$ ,  $\gamma_o = 100 \text{ pcf}$ ,  $\gamma_{100} = 140 \text{ pcf}$ ,  $\gamma_{80} = 113 \text{ pcf}$ , (Sh. A-3)

Sat. NWC = 18% (Sh. A-3),  $\gamma_{sat} = 113 \times 1.18 = 133 \text{ pcf}$

$$\gamma_{buoyant} = 133 - 62.4 = 70.6 \text{ pcf}$$

\* See Sh. A-4

For Wall Design (U Wall) use  $K_0$  (EM 1110-2-2502 4.6(4), p 5)

For  $\phi = 30^\circ$ ,  $K_0 = 1 - \sin \phi$ ,  $K_0 = 1 - 0.5 = 0.5$

## Design Lateral Earth Pressures for Box.

1) Saturated Unit Wt.

$$p_o = 133 \times 0.5 = 66.5 \text{ pcf/ft.}$$

2) Buoyant Unit Wt.

$$p_b = 70.6 \times 0.5 = 35.3 \text{ pcf/ft.}$$

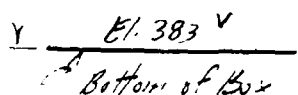
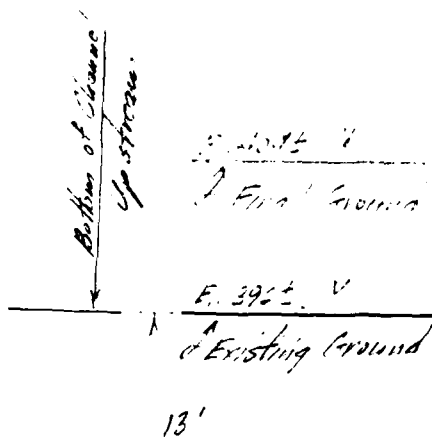
Active Lateral Earth Pressures for Wingwalls  $\text{for } \phi = 30^\circ$

1) Saturated  $p_a = 133 \times 0.333 = 44.3 \text{ pcf}$

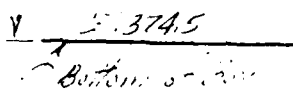
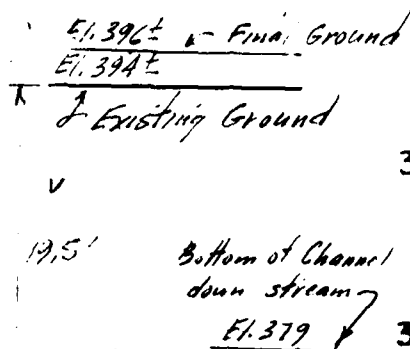
$K_0 = 0.333$

2) Buoyant  $70.6 \times 0.333 = 23.5 \text{ pcf}$

85

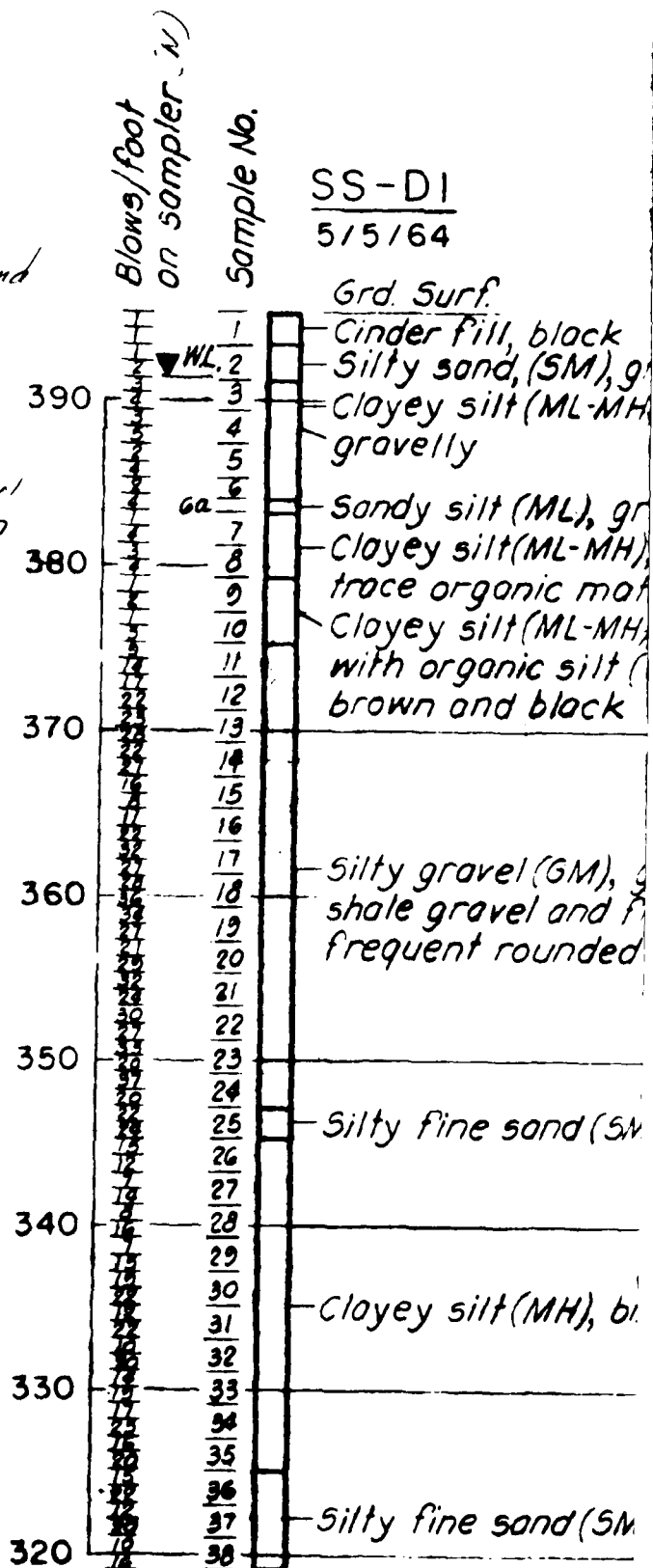


BOX NO. 1



BOX NO. 2

ELEVATIONS IN FEET



SS-D

4/30/64

Blows / foot  
on sampler

Sample No.

Grd. Surf.

▼ W.L.

black  
(SM), gravelly, brown  
(ML-MH), brown

wet

(ML), gray  
(ML-MH), gray,  
mic matter  
(ML-MH), interbedded  
ic silt (OL),  
black

moist

(GM), gray, with  
el and fragments  
ounded pebbles

wet

sand (SM), brown, FW

(MH), brown gray

wet

sand (SM), moist, gray

1	Cinder fill, black	
2	Gravelly clay (CL), brown, wet,	
3	with cinders and slag	
4	Clayey silt (ML-MH), brown, moist	
5		
6	Silty clay (CL), brown gray, moist,	
7	thin peat seams	
8		
9	Clayey silt (MH), gray, with	
10	seams of fine gravel	
11	Sandy gravel (GP), yellow,	
12	trace of clay	
13		
14	Sandy gravel (GP), brown,	wet
15	trace of clay	
16		
17	Silty fine sand (SM) yellow	
18	Sandy gravel (GP), gray, brown	
19	trace of silt	
20	Silty clay (CL-CH), red, moist	
21	Clay (CL), varved	
22	boulder	
23	Clay (CH)	
24	Clay (CL), varved	
25		
26	Clay (CL)	
27		
28	Clayey silt (CL-ML)	wet, gray
	Clay (CL)	
	Clayey silt (CL-ML)	
	Clay (CL)	
	Silty clay (CL-ML)	

5.4-2

SS-1  
5/14/62

Blows/foot on sampler		Sample No.		
▼ W.L.			Grd. Surf.	
8	10	1	Topsoil	
17	19	2	Clay (CL), brown, moist	
24	26	3	Silty clay (CL), gray, with	
26	28	4	organic, wet	
19	21	5		
14	16	6		
1	3	7	Fine sandy silt (ML), with	
2	4	8	wood chips and peat	wet,
10	12	9		gray
16	18	10	Silty clay (CL-ML)	
25	27	11	Silty clay (CL-CH)	
47	49	12	Clayey gravel (GC),	
51	53	13	yellowish	
27	29	14	Clay, sand and gravel (GC),	
36	38	15	yellowish	
23	25	16	Silty fine sand (SM)	
20	22	17		
2	4	18	Clay (CL-CH)	
7	9	19	Silty fine sand (SM)	wet,
8	10	20		brown
11	13	21	Silty clay (CL-CH)	
12	14	22	Sandy gravel (GM), silty	
81	83	23	Clay, sand and gravel (GC)	
42	44	24	Silty sand (SM), gravelly,	
40	42	25	grayish	
38	40	26	Sandy gravel (GP)	
37	39	27		
44	46	28		
29	31	29		
33	35	30		
24	26	31		
43	45	32		
33	35	33		
34	36	34		
15	17	35		
19	21	36		
19	21	37		
18	20	38		
16	18	39		
16	18	40		
16	18	41		
18	20	42		

Notes:

For location see Sh. A-26  
For location of borings  
see Sh. A-26



LEGEND - (SUBSURFACE EXPLORATIONS)

5/27/64 - DATE EXPLORATION WAS COMPLETED

▼ W.L. - WATER LEVEL IN HOLE WHEN EXPLORATION WAS MADE

150(0.4') - BLOWS PER FRACTION OF FOOT AS INDICATED

307 - MOISTURE CONTENT, % DRY WEIGHT

FW - FREE WATER IN JAR SAMPLE

SP - SAND, POORLY GRADED

SM - SAND, SILTY

SC - SAND, CLAYEY

GC - GRAVEL, CLAYEY

GM - GRAVEL, SILTY

GP - GRAVEL, POORLY GRADED

GW - GRAVEL, WELL GRADED

ML - INORGANIC SILT, LOW TO NO PLASTICITY

MH - INORGANIC SILT, HIGH PLASTICITY

ML-MH - SILT, BORDERLINE OR MEDIUM PLASTICITY

CL-ML - BORDERLINE BETWEEN CLAY AND SILT

SM-ML - BORDERLINE BETWEEN SILTY SAND AND SANDY SILT

CL - INORGANIC CLAY, LEAN, LOW TO MEDIUM PLASTICITY

CH - INORGANIC CLAY, FAT, HIGH PLASTICITY

CL-CH - BORDERLINE BETWEEN LEAN AND FAT CLAY

OL - ORGANIC SILT OR CLAY, LOW TO MEDIUM PLASTICITY

PT - PEAT, PREDOMINANTLY ORGANIC

N - NO BLOW COUNT OR SAMPLE TAKEN

P - SAMPLER PUSHED BY HAND

W - SAMPLER SANK UNDER WEIGHT OF RODS AND HAMMER ALONE

NR - NO RECOVERY

———→ - INDICATES THE APPROXIMATE CHANNEL GRADE.

AD-A101 711

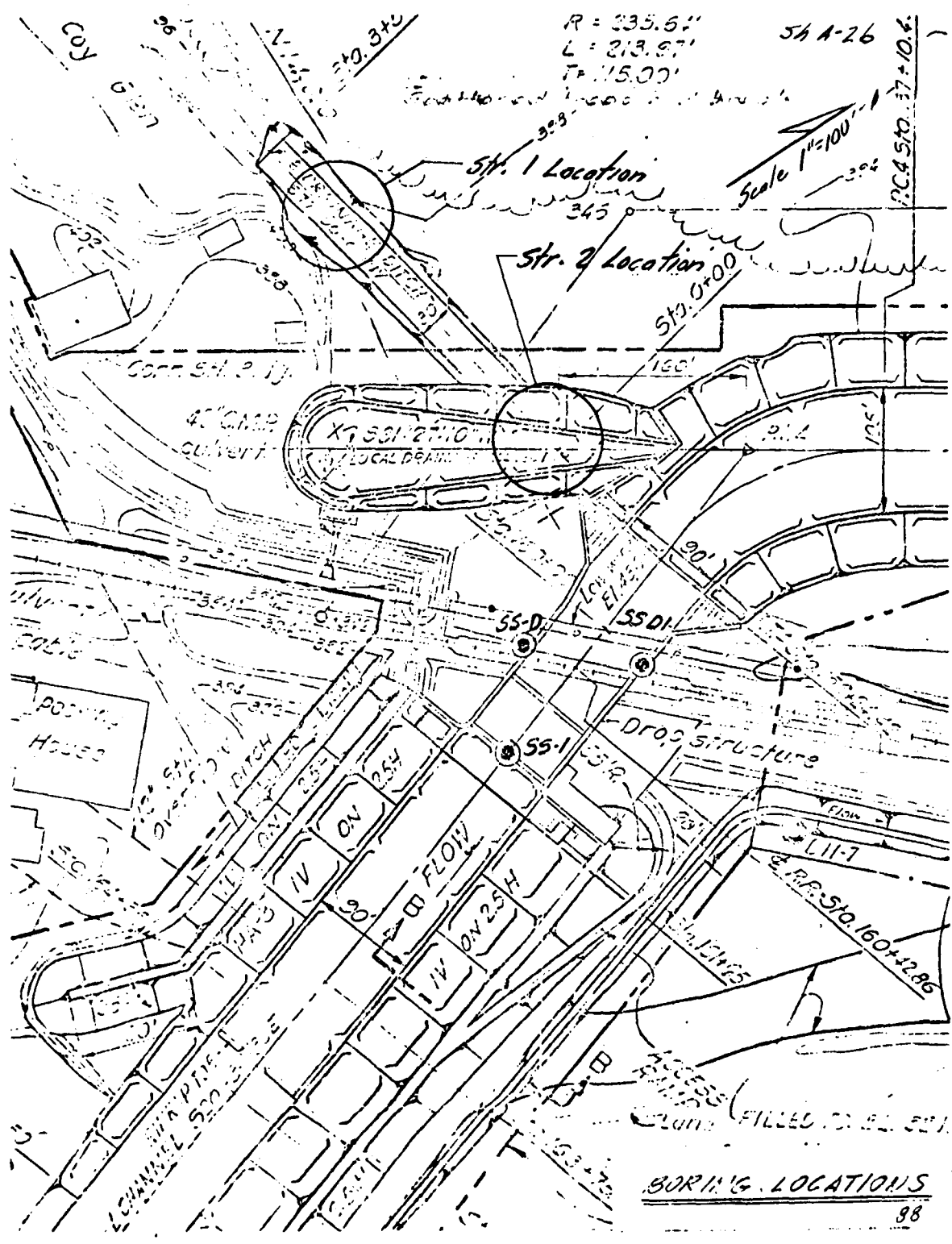
HOWARD NEEDLES TAMMEN AND BERGENOFF NEW YORK  
ENERGY DISSIPATOR FACILITIES AND RIPRAP REPAIR,  
AUG 75

F/G 13/134  
COY GLEN AND CA--ETC (1)  
DACW49-75-C-0052

UNCLASSIFIED

44

2 OF 3  
AD  
A 10111



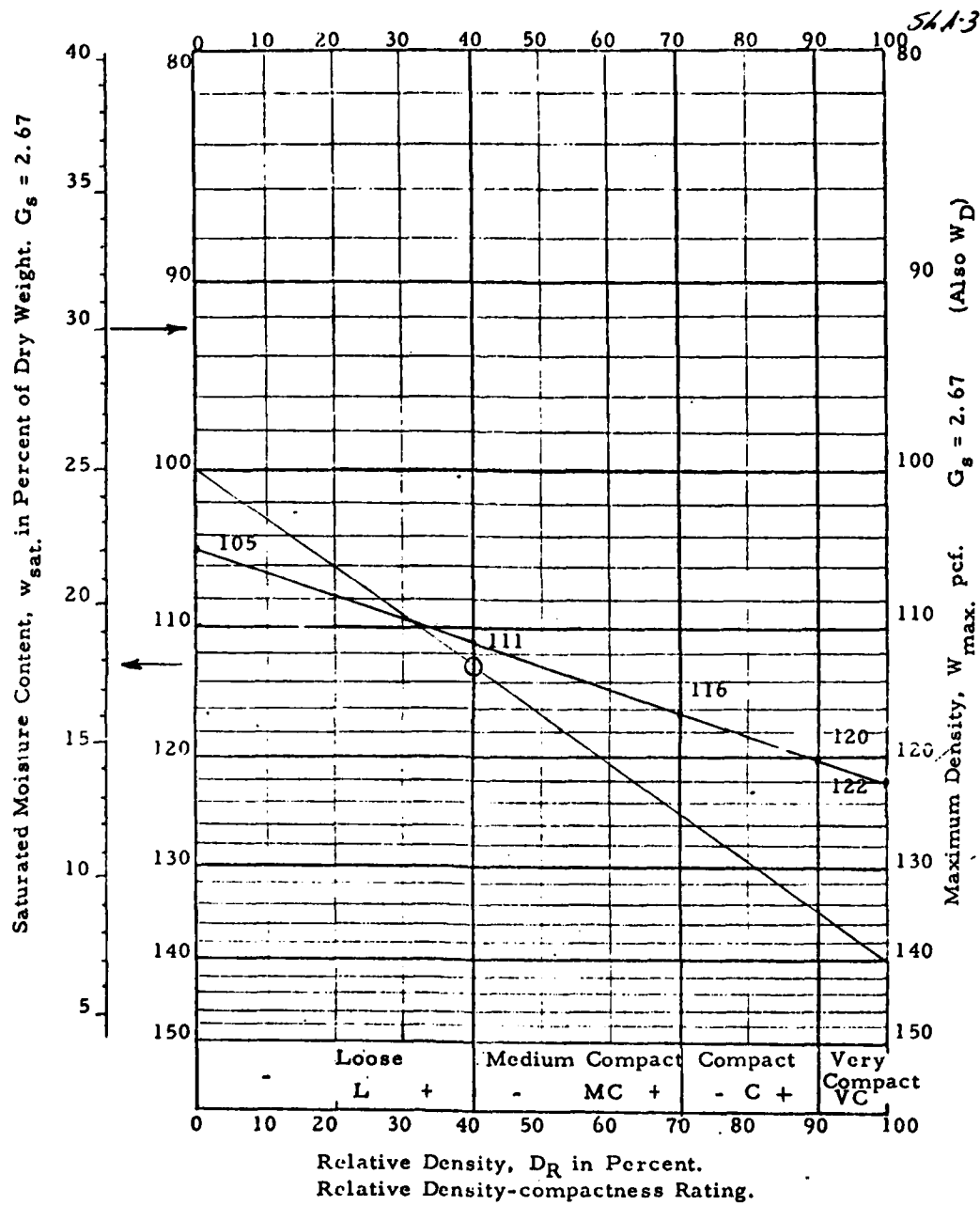
R = 335.6'  
 L = 213.6'  
 T = 15.00'

54 A-26

Scale 1"=100'

PC4 STA. 17+10.6

BORING LOCATIONS



$$D_R = \frac{e_L - e_N}{e_L - e_D} 100\% = \frac{1/W_L - 1/W_N}{1/W_L - 1/W_D} 100\% = \frac{w_L - w_N}{w_L - w_D} 100\%$$

FIG. 2 - Relative Density, Unit Weight, and Compactness Rating Diagram.

# HNTB

CALCULATIONS FOR

Coy Glen, Ithaca, N.Y.

MADE BY AAS-J

DATE 4-2-75

JOB NO. 4204

CHECKED BY POJ

DATE 5-6-75

SEC. NO.

SHEET NO. A-4

Unit Wt. of GP, GM soil (Sh. A-1)

$\gamma_{max} = \gamma_{100} = 135-145 \text{ pcf}$  (Table B1, Appendix B, "the Unified  
Use  $\gamma_{100} = 140 \text{ pcf}$  Soil Classification System, WES  
TM 3-357, 1957)

$\gamma_{min} = \gamma_0 = 92 \text{ to } 115 \text{ pcf}$  (Nos. 4-11, Table II "Physical, Stress-  
Strain and Strength Responses of  
Use  $\gamma_0 = 100 \text{ pcf}$  Granular Soils" by D. M. Burmister,  
ASTM STP No. 322, 1962)

For  $N=23$ ,  $D_r = 40\%$  (Fig. 6 D.M. Burmister reference)  
also Sh. A-9

HOWARD NEEDLES TAMMEN & BERGLUND  
CONSULTING ENGINEERS

# HNTB

CALCULATIONS FOR

Coy Glen, Ithaca, N.Y.

MADE BY AAS-J DATE 4-25-75 JOB NO. 4024  
 CHECKED BY 101 DATE 5-6-75 SEC. NO.   
 SHEET NO. A-5

~~Estimate~~ Check Strength of Clay Soils

	<u>Boring SS-D1</u>	<u>Boring SS-D</u>
Elev 395 to 393	EN=33, or N=33/13=2.5	EN=33, or N=33/13=2.5
395 to 377±	48, 48/19=2.5	46, 46/18=2.5

From Terzaghi & Peck Table 45.2

for  $N=2$   $q_u = 0.25 \text{ tsf}$  or  $c = 250 \text{ psf}$  or  $c = 125 \text{ psf}$  N

Est.  $c = 125 \times 2.5 = 312 \text{ psf}$

note: varied clays from Hockensack Members  
 min  $c = 500 \text{ psf}$  for  $N = \text{push}$

Critical Depth of Excavation

For S.F. = 1.5  $c_d = 312 / 1.5 = 208 \text{ psf}$

For saturated clay use  $\gamma_s = 121 \text{ pcf}$  (Sh. A-1)

Check for HiW and 1:1 excavation slopes, see Sh. A-6

$L =$	63.5'	45'
$S_N =$	0.197	0.170
$H_c =$	8.7'	10.1'

$$H_c = \frac{c_d}{\gamma_s} = \frac{208}{121 S_N} = \frac{1.72}{S_N}$$

Note: Depth of Excavation: Box No. 1 = 16'  
 No 2 = 20'

Check for 2H:1V,  $i = 26.5^\circ$ ;  $S_N = 0.153$ ,  $H_c = 11.3'$

Check Bearing Capacity of Clay Soil (above Elev. 375)

$q = 5.14 c = 5.14 \times 312 = 1600 \text{ psf}$  (no surcharge)

surcharge effect;  $D_f = \text{El. } 396 - 383 = 13'$

use  $\gamma \times \gamma_s = 121 \times 62.4 = 58 \text{ pcf}$

$D_f \gamma = 13 \times 58 = 755 \text{ psf}$

Total  $q' = q + D_f \gamma = 1600 + 755 = 2355 \text{ psf}$

\* Ref. Terzaghi & Peck 2nd Ed., page 222

Box: max load (Sh. 37) 2.0 ksf back, 0.66 ksf front  
 normal D.L. 0.50 ksf

Wells: to a (Sh. 44) 3.87 ksf

HOWARD NEEDLES TAMMEN & BERENSON CONSULTING ENGINEERS

HOWARD NEEDLES TAMMEN & BERENSON CONSULTING ENGINEERS

# HNTB

CALCULATIONS FOR

Coy Glen, Ithaca, N.Y.

MADE BY AAS-J DATE 5-3-75 JOB NO. 2024  
CHECKED BY COU DATE May 7-75 SEC. NO.   
SHEET NO. A-5a

## Revised Critical Depth of Excavations

For new value of  $C = 600 \text{ psc}$  (Sec 56, A-T)  
For  $SF = 1.5$ ,  $C = 600 \text{ psc}$ ,  $C_d = \frac{600}{1.5} = 400 \text{ psc}$

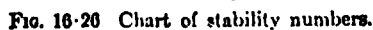
Slope	2H:1V	1H:1V	2H:1V
$i'$	68.5°	45°	26.5°
$S_N$	0.197	0.170	0.153
$H_c$	16.8'	19.5'	21.7'

$$H_c = \frac{C_d}{7 S_N} = \frac{400}{121 S_N} = \frac{3.32}{S_N}$$

Note: Depth of Excavations Box No. 1 = 16'  
Box No. 2 = 20'

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HOWARD NEEDLES TAMMEN & BERENDSON



$$H_c = \frac{C_d}{\gamma S_N}$$

$$SF = \frac{C_e}{C_d}$$

Ref. D. W. Taylor, "Fundamentals of Soil Mechanics",  
John Wiley & Sons, Inc. 1948.



# HNTB

CALCULATIONS FOR

Coy Glen, Ithaca, N.Y.

MADE BY AAS-J DATE 5-1-75 JOB NO. 4204  
 CHECKED BY J.A.P. DATE 5-10-75 SEC. NO.   
 SHEET NO. A-7

## Evaluation of Clay Shear Strength

From pocket penetrometer and terrane device used at site on May 1, 1975

- 1) Coy Glen Sta. 1+50 south bank, stream bed  
 @ El. 389 ±

a)  $T_v = 0.48, 0.40, 0.40$ ,  $p_p = 1.0, 1.5, 1.5, 1.0$  (at water level) ✓

b)  $T_v = 0.53, 0.57$  ✓,  $p_p = 2.0, 2.0, 1.75$  (above water level) ✓

north bank of stream, above water level ✓

$p_p = 1.75, 1.5, 1.5$  ✓

c) Sta. 1+85 south bank at water level ✓

$T_v = 0.4, 0.4$ ,  $p_p = 1.5, 1.25, 1.5, 1.5$  ✓

$T_v$  readings = cohesion strength test ✓

$p_p$  = unconfined strength (2xc) test ✓

## Comments

- 1)  $T_v$  tests indicate  $c \geq 800 \text{ psf}$  ✓  
 $p_p$  " "  $c \geq 1000 \text{ psf}$  ✓

- 2) From Sta. A-2  $N$  values @ El. 389 or 3.5 where ✓  
 as a lower point at  $N = 2.5$  it may be that ✓  
 the shear strength applies at this level (El. 389) ✓  
 may be in a denuded zone.

Adjustment for  $N$  ✓ ✓ ✓  
 $\text{est. } c \geq \frac{2.5}{3.5} \times 800 = 570 \text{ psf}$

USE  $c = 600 \text{ psf}$  ✓

## Revised Bearing Capacity (Ref. Sta. A-5)

$$q = 5.14c + D_f \gamma = 5.14 \times 600 + 13 \times 58 = 3084 + 750 = 3830 \text{ psf}$$

## Drop Structure Foundation Load

Dead Load 0.96 ksf ✓ (Str. Calc. Sta. 37A)

DL + Live Load 2.95 " max. ✓

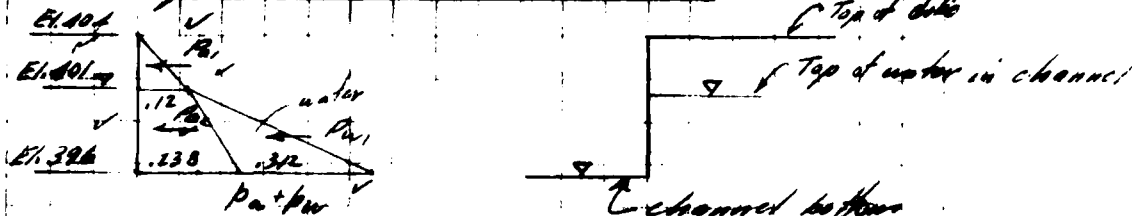
Walls: high wall @ toe = 2.07 " ✓ " 44

low " @ " 2.42 " ✓ " 51

24

## Check for Sheet Pile Retaining Walls

### 1. Upstream End of Box Pier



- a) For Sand Backfill assume  $\phi = 30^\circ$ ,  $K_a = 0.333$  ✓  
 El. 404 to 401  $\gamma_{\text{moist}} = 120 \text{ pcf}$  ✓  $p_a = 0.333 \times 120 = 40 \text{ pcf/ft}$  ✓  
 El. 401 to 396  $\gamma_{\text{moist}} = 133 \text{ pcf (Sch. A-1)}$  ✓  $\gamma_{\text{buoy}} = 133 - 62.4 = 70.6 \text{ pcf}$  ✓  
 ✓  $p_a = 0.333 \times 70.6 = 23.5 \text{ pcf/ft}$  ✓  
 @ El. 401  $p_a = 8 \times 40 = 120 \text{ psf}$  ✓ ✓ ✓  
 396  $= 120 + (5 \times 23.5 = 117.5) = 238 \text{ psf}$  ✓ ✓ ✓ USE CONTROLS  
 $p_w = 5 \times 62.4 = 312$  ✓

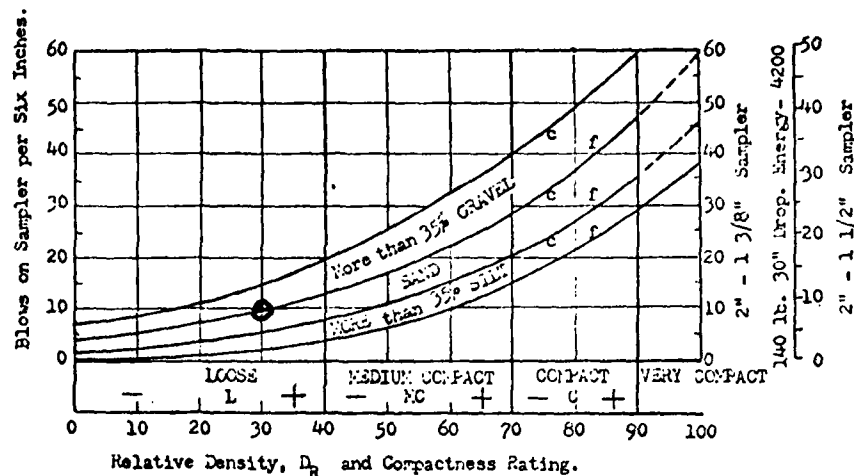
- b) For Clay Soil  $\gamma_{\text{moist}} = 121 \text{ pcf (Sch. A-1)}$  ✓  
 $\gamma_{\text{buoy}} = 121 - 62.4 = 58.6 \text{ pcf}$  ✓  
 $C = 600 \text{ psf (Sch. A-7)}$  ✓  
 $p_a = \gamma h - 2C$  ✓  
 @ El. 396  $p_a = (3 \times 121 = 363) + (5 \times 58.6 = 293) - (2 \times 600 = 1200)$  ✓  
 $= 656 - 1200 = -544$  ✓

- c) Resistance Below Elev. 396 - Clay Soil  
 level at which, active pressure of clay  $> 0$   
 $h = 2C / \gamma_a = \frac{1200 - 363}{58.6} = 9.3' \text{ El. } 386.7$  ✓

- d) Assume differential water head, El. 401 behind sheeting,  
 El. 396 in front of sheeting reduces to zero at El. 375.  
 sand below Elev. 375 should provide equal water  
 head on each side of sheeting below this level.

Active pressure in clay @ El. 375 ✓  
 $p_a = (3 \times 121 = 363) + (26 \times 58.6 = 1524) - 1200 = 687 \text{ psf}$  ✓

\* Fig. 10-1 Ref. (1) "Design Manual, Soil Mechanics, Foundations, and Earth Structures, DM-7" U.S. Navy NAVFAC, Mar., 1971. ✓



Approximate adjustment of blows per six inches,  $E'$  for new weight of hammer,  $W$  and height of drop,  $h$  and new outside and inside diameters of sampler,  $D_o$  and  $D_i$ .

$$\text{New Scale of Blows/6"} \quad B = B' \times \frac{4200}{WH} \times \left[ \frac{D_o^2 - D_i^2}{2.0^2 - 1.375^2} \right]$$

FIG. 6.—Compactness Performance Rating for Evaluation of In-Place Relative Densities and Compactness from Boring Records and Blow-Counts on a 2 by 1 1/2-In. Sampler under a 140-lb Hammer Falling 30 in. Blow-counts are governed by relative density in the sampling depth and by the influences of coarseness or fineness of soils sampled. (4, Figs. 4 and 5, pp. 1257-1258.)

TABLE III.—COMPACTNESS PERFORMANCE RATING FOR EVALUATION OF BLOW-COUNTS ON A 2 BY 1 1/2-IN. SAMPLER UNDER A 140-LB HAMMER FALLING 30 IN.

Relative Density, $D_R$	0	20	30	40	50	60	70	80	90
		LOOSE			MEDIUM COMPACT			COMPACT	VERY COMPACT
		L	+		MC	+		C	VC
More than 35% GRAVEL	7.3	11.4	14.6	19.2	25.7	32.1	39.0	45.0	51.2
SAND	4.4	7.3	9.4	12.3	17.0	22.8	29.2	33.0	39.5
More than 35% SILT	1.5	3.7	5.7	7.9	10.8	14.6	20.5	24.0	29.3
	0.7	1.2	2.3	3.2	5.9	9.4	14.6	18.3	23.5

(Penetration Resistance in blows/6 Inches)

Re: D. M. Burmister, "Physical, Stress-strain and Strength Responses of Granular Soils," ASTM STP No. 322, 1962.

SA. A-1.

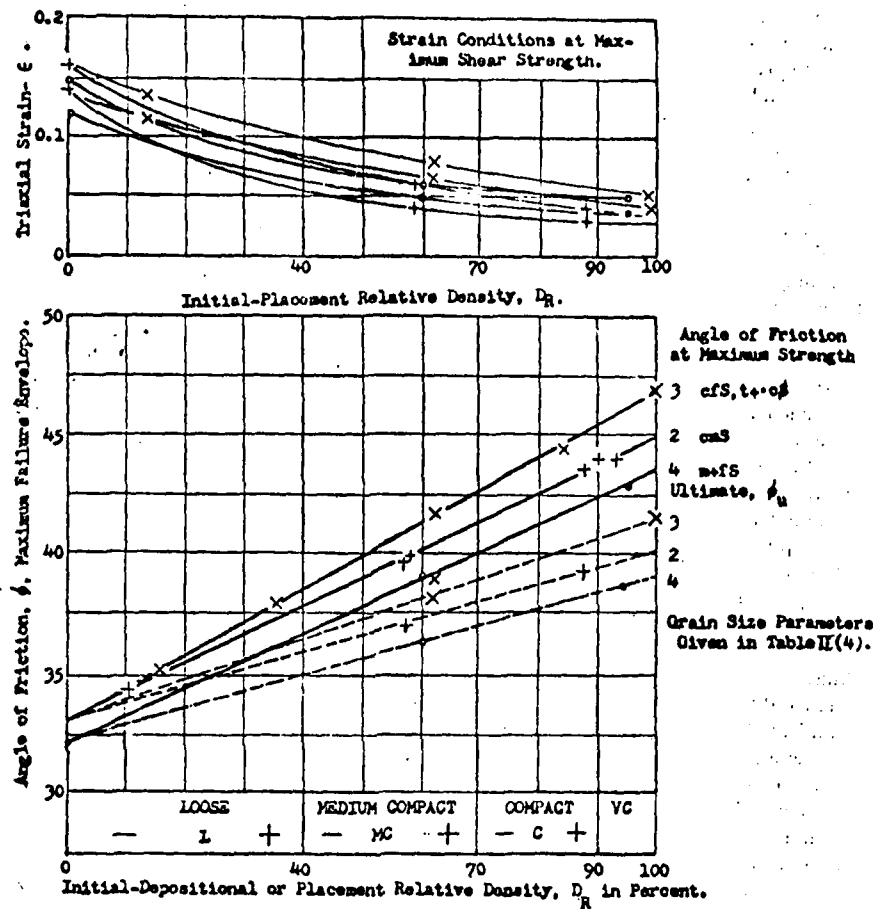


FIG. 14.—Angle of Friction Performance Rating Showing Controlling Influences of Identification and Relative Density of Granular Soils and of Strain Conditions on the Simultaneously Mobilizable Shearing Strengths. (S. Raamot test data.) The angle of friction must be referenced to the initial-depositional relative densities of Fig. 5 as a tentative basis.

Ref. D. M. Burmister, "Physical, Stress-Strain and Strength Responses of Granular Soils", ASTM STP No. 322, 1962.

# HNTB

CALCULATIONS FOR

Loy Glen, Ithaca, N.Y.

MADE BY NAS-T DATE 5-3-75 JOB NO. 4208  
 CHECKED BY 727 DATE 5-10-75 SEC. NO.   
 SHEET NO. A-11

c) For Sand and Gravel Layer, El. 375 to 360 ✓

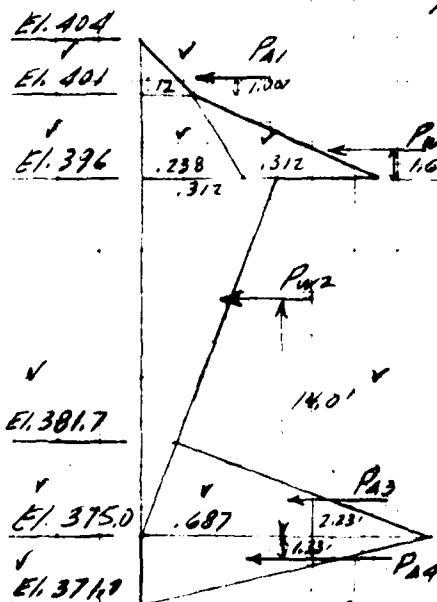
at N=20 (Boring SS-D1) Sh. A-2 ✓  
 21 ( " SS-D) " ✓

For  $\phi = 30^\circ$ ,  $D_r = 30\%$  (Sh. A-9) ✓  
 for  $D_r = 30\%$ ,  $\phi = 35^\circ$  (Sh. A-10) ✓  
 $\gamma_s = 133 \text{ pcf}$  (Sh. A-1) ✓  
 $\gamma_b = 70.6$  ( " ) ✓

For  $\phi = 35^\circ$   $K_A = \tan^2(45 - \frac{\phi}{2}) = \tan^2(45 - \frac{35}{2}) = \tan^2 27.5^\circ = 0.26$  ✓

$K_p = 1/K_A = 3.85$  ✓

for S.F. = 1.5  $\frac{K_p - K_A}{1.5} = \frac{3.85 - 0.26}{1.5} = 2.4$  ✓  
 $\frac{7(K_p - K_A)}{1.5} = \frac{7(3.85 - 0.26)}{1.5} = 170 \text{ psf/H.}$  ✓



$P_1$  arm  $M_{E1.396}$   
 $(12 \times 5 = 0.60) \frac{2.5}{3} = 1.500$  ✓  
 $(2 \times 1.13 \times 5 = 0.395) \frac{5}{3} = .492$  ✓  
 $.995$   $1.992$  ✓  
 $\bar{x} = 1.992 \div .995 = 2.01'$  ✓

Point  $P_4 = 0$  ✓  
 $\bar{x} = .687 \div 0.170 = 4.04'$  El. 371.0 ✓

Elements @ El. 371.0

$P_i$	$R_w$	arm	$M_{E1.371.0}$
$P_{A1}$	$(\frac{1}{2} \times 3 \times 12) =$	0.180	31.0
$P_{A2}$	$(\frac{1}{2} \times (12 + 2.38) \times 5) =$	0.896	27.01
$P_{W1}$	$(2 \times 3.12 \times 5) =$	0.780	26.67
$P_{W2}$	$(2 \times 3.12 \times 21) =$	3.280	18.0
$P_{A3}$	$(2 \times 6.87 \times 6.7) =$	2.300	6.23
$P_{W3}$	$(2 \times 6.87 \times 4.04) =$	1.388	2.67
$\Sigma P_A$	8.824		$M_o = 127.69$
$\bar{x}$	$127.69 \div 8.824 =$	14.48'	

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# HNTB

CALCULATIONS FOR

Coy Glen, Ithaca, N.Y.

MADE BY

AAS-J

DATE

6-19-75

JOB NO.

4204

CHECKED BY

JhJ

DATE

6-27-75

SEC. NO.

SHEET NO.

A-26

Note: Sheets A-12 to  
A-15 VOID

Passive pressure in clay - front of sheeting

$$p_p = \gamma h + \frac{2c}{3F} \quad \text{for } c = 0.6 \text{ ksf, } SF = 1.5, \quad \checkmark$$

$$\text{@ El. 376 } \gamma h = 0 \quad \checkmark$$

$$p_p = 0 + \frac{2 \times 0.6}{1.5} = 0.80 \text{ ksf} \quad \checkmark$$

$$\text{@ El. 386 } h = 10', \gamma = 0.0586; \gamma h = 10 \times 0.0586 = 0.586 \quad \checkmark$$

$$p_p = 0.586 + 0.80 = 1.386 \text{ ksf} \quad \checkmark$$

Point at which active pressure = 0

$$h = \frac{2c}{\gamma} = \frac{2 \times 0.6}{0.0586} = 20.8' \text{ El. 375.5} \quad \checkmark$$

Passive pressure in clay - behind sheeting

$\gamma h$  (for sand above El. 396)  $\checkmark$

$$\text{@ El. 396 } \gamma h = (3 \times 120 = 360) + (5 \times 0.0706 = 0.353) = 1.13 \quad \checkmark$$

$$p_p = \gamma h + \frac{2c}{3F} = (0.713 + \frac{2 \times 0.6}{1.5} = 0.80) = 1.513 \text{ ksf} \quad \checkmark$$

$$\text{@ El. 386 } p_p = 1.513 + (10 \times 0.0586 = 0.586) = 2.099 \text{ ksf} \quad \checkmark$$

point at which active pressure = 0

$$h = \frac{2c - 0.713}{\gamma} = \frac{2 \times 0.6 - 0.713}{0.0586} = 8.3' \text{ El. 387.7} \quad \checkmark$$

Water Pressure below El. 396

$$\text{@ El. 396 } p_w = 5 \times 62.4 = 312 \text{ psf} = 0.312 \quad \checkmark$$

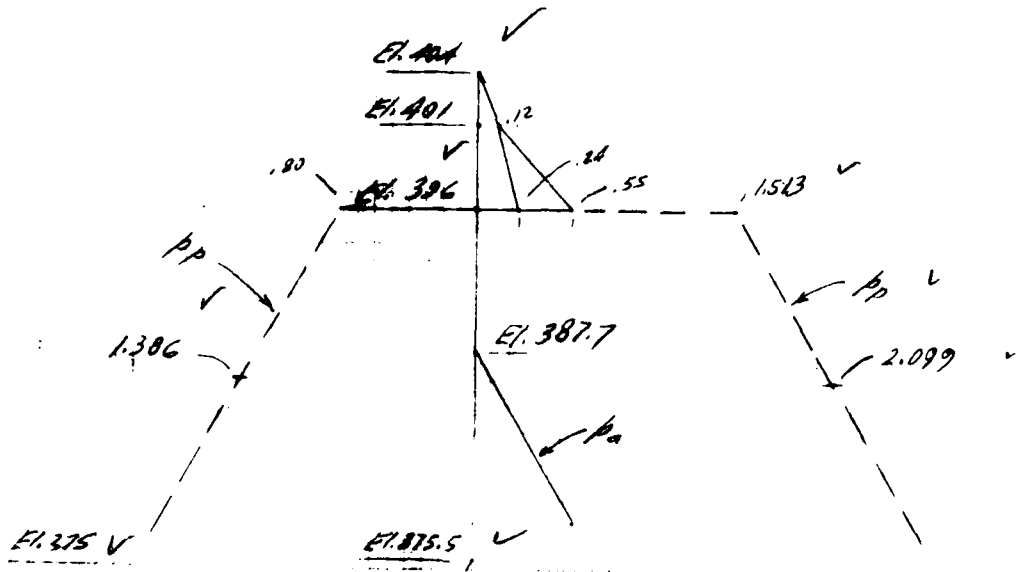
assume  $p_w = 0$  at bottom of sheeting (diff. in head from front to back is zero).

Above data plotted on sh. A-27

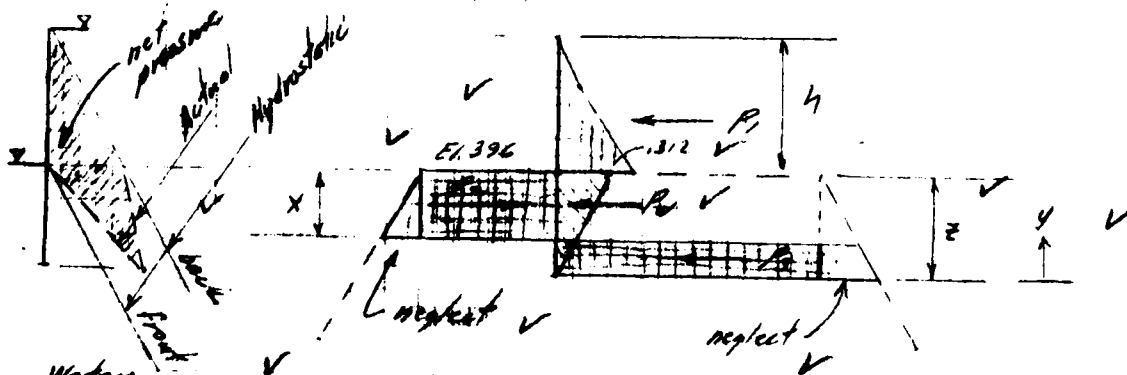
		arm	M <sub>996</sub>
$P_{a1}$	$6 \times 3 \times 12 =$	180	6.00
$P_{a2}$	$0.12 \times 5 =$	600	2.50
	$2 \times 5 (1.238 - 12 = 1.18) =$	295	1.67
$P_w$	$2 \times 5 \times 0.624 =$	780	1.67
$P_i$		1.855	4.376

CALCULATIONS FOR Coy Glen, Ithaca, N.Y.

BACK CHECKED BY \_\_\_\_\_ DATE \_\_\_\_\_



$p_p = \gamma h + 2C$  Ref: B.K. Hough "Basic Soil Engineering" Ronald Press, 1957 Pg. 9-23  
 $p_a = \gamma h - 2C$  Note: use  $p_a \geq 0$  (for  $2C > \gamma h$ ,  $p_a = 0$ )



Water Pressure

$\Sigma H = P_1 - P_2 + P_3 + P_w = 0$

$P_2 = \gamma \left( \frac{1}{2} \gamma h + 2C \right) \cdot x(2C) = 0.80x$

$\hookrightarrow$  neglect to simplify equations

$P_3 = (\gamma h + 2C)(z-y) + \frac{1}{2} \gamma (y^2 - z^2)$

$\hookrightarrow (\gamma h + 2C)(z-y) = 1.513(z-y)$

**HNTB**

CALCULATIONS FOR

Coy Glen, Ithaca, N.Y. ✓

 MADE BY AAS-J DATE 6-20-75 JOB NO. 4204  
 CHECKED BY J.K.J DATE 6.27.75 SEC. NO. \_\_\_\_\_  
 SHEET NO. A-28

To simplify analysis assume passive pressure in clay below Elev. 396 is constant with depth - see diagram sk. A-27 ✓

Forces (Ref. sk. A-27) ✓

$$P_1 = 1.855 \text{ k} \quad \checkmark$$

$$P_2 = .80 \text{ k} \quad \checkmark$$

$$P_3 = 1.513 (z - x) \quad \checkmark$$

$$P_u = 1(1.312)z = .156z \quad \checkmark$$

$$\Sigma H = 0; \quad P_1 - P_2 + P_u + P_3 = 0 \quad \checkmark$$

$$1.855 - .80z + 1.513z - 1.513z + 0.156z = 0 \quad \checkmark$$

$$2.313z = 1.855 + 1.669z \quad \checkmark \quad z = .802 + .722z$$

$$x^2 = 0.643 + 1.158z + 0.521z^2 \quad \checkmark$$

Moments C/E 396 (Ref. sk. A-27) ✓

$$M_1 = -4.376 \quad \checkmark$$

$$M_2 = -P_2 \left(\frac{z}{2}\right) = 0.40z^2 \quad \checkmark$$

$$M_3 = 1.513z \left(\frac{z}{2}\right) - 1.513x \left(\frac{z}{2}\right) = .757z^2 - .757xz^2 \quad \checkmark$$

$$M_u = P_u \left(\frac{z}{3}\right) = 0.052z^2 \quad \checkmark$$

$$\Sigma M_0 = 0; \quad -M_1 - M_2 + M_3 + M_u = 0 \quad \checkmark$$

$$-4.376 - 0.40z^2 + 0.757z^2 - 0.757xz^2 + 0.052z^2 = 0 \quad \checkmark$$

$$-4.376 - 1.157xz^2 + 0.809z^2 = 0 \quad \checkmark$$

$$-4.376 - .744z + 1.340z^2 - 0.603z^2 + 0.809z^2 = 0 \quad \checkmark$$

$$-5.120 - 1.840z + 0.206z^2 = 0 \quad \checkmark$$

$$z^2 - 6.50z - 24.85 = 0 \quad \checkmark$$

$$z = \frac{6.50 \pm \sqrt{(42.25 + 99.40)}}{2} = 14.165 \quad \checkmark$$

$$= \frac{6.50 \pm 11.20}{2} = \frac{17.70}{2} = 8.8 \quad \checkmark$$

Use 9.5' 101  
typ Elev. 386.5

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CALCULATIONS FOR

Coy Glen, Ithaca, N.Y.

MADE BY AAS-J DATE 6-20-75 JOB NO. 4204  
CHECKED BY J.K.J DATE 6.27.75 SEC. NO. \_\_\_\_\_  
SHEET NO. A-29

$$x = 0.902 + 0.722(9.2) = 0.902 + 6.642 = 7.44'$$

Point of zero shear  $y$  measured up from bottom of sheeting  
 $P_3 + P_u - P_2 = 0$ 

$$P_3 = 1.513(z - x) = 1.513(9.2 - 7.44 = 1.76) = 2.663$$

$$P_u = \frac{1}{2} (10.312) \frac{y}{2} = 0.017y^2$$

$$P_2 = 0.80(y - 1.76)$$

$$2.663 + 0.017y^2 - 0.80y + 1.408 = 0$$

$$0.017y^2 - 0.8y + 4.071 = y^2 - 47.06y + 239.47 = 0$$

$$y = \frac{+ 47.06 \pm (2214.64 - 957.88 - 1256.76)^{1/2}}{2} = \frac{47.06 \pm 35.45}{2}$$

$$y = \frac{11.61}{2} = 5.805'$$

$$\text{Mom @ } y = 5.81'$$

$$M = P_3[y - (\frac{z-x}{2})] + P_u(\frac{y}{3}) - P_2(\frac{y \cdot z + x}{2})$$

$$= 2.663[5.81 - \frac{1.76}{2}] + 0.017(5.81)^2(\frac{5.81}{3}) -$$

$$0.80(5.81 - 1.76)(\frac{5.81 - 1.76}{2})$$

$$= 13.13 + 1.11 - 6.56 = 7.68 \text{ k'}$$

$$\text{for } f_c = 18 \text{ ksi, req'd SM} = \frac{12 \times 7.68}{18} = 5.12 \quad \text{PMA 22} \\ \text{USE MP115} \\ \text{SM} = 5.4$$

$$\text{Total Length} = 8 + 9.5 = 17.5'$$

2. Downstream end Box No. 1

PMA 22

USE MP115

$$\text{Total Length} = 17.5'$$

# HNTB

CALCULATIONS FOR

Coy Glen, Ithaca, N.Y.

MADE BY AAS-J DATE 6-20-75 JOB NO. 4204  
 CHECKED BY J.H.J DATE 6.27.75 SEC. NO.   
 SHEET NO. 4-30

## 3. Upstream End of Box No. 2

El. 395.5 - 17.5 = El. 378.0 sheeting above sand layer @ El. 375 PMA 28

USE MP 115 Total Length = 17.5'

## 4. Downstream End of Box No. 2

El. 386

Final Ground

El. 383

El. 379

bottom of channel

El. 375

bottom of clay stratum

El. 386

El. 383

El. 379

$P_a$   
12  
 $P_a$   
214

For sand backfill above El. 379

@ El. 383  $p_a = 0.12$  (SL-A-8)

@ El. 379  $p_a = 0.12 + 4(0.0706) = 0.333$

$= 0.12 + 0.094 = 0.214$

Note: water level @ El. 383 both sides of sheeting

		P	arm	M
$P_1$	$1 \times 3 \times 12 =$	0.180	5.0	0.900
$P_2$	$4 \times 12 =$	0.480	2.0	0.960
$P_3$	$2 \times 4 \times 0.094 =$	0.188	2/3	0.251
		0.848		2.111

## a. Design assuming all clay below El. 379

Passive pressure in clay - front of sheeting @ El. 379

$p_p = 0.80 \text{ ksf}$  (See SL-A-26)

Passive pressure in clay - behind sheeting @ El. 379

$p_p = \gamma H + \frac{2c}{3}$

$\frac{2c}{3} = \frac{2 \times 0.6}{3} = 0.40$

$\gamma H = (3 \times 120 = 360) + (4 \times 0.706 = 2.82) = 0.642$

$p_p = 0.642 + 0.40 = 1.042 \text{ ksf}$

**HNTB**

CALCULATIONS FOR

Log Glen, Ithaca, N.Y.

MADE BY AA5-T  
CHECKED BY J27DATE 6-20-75  
DATE 6-27-75JOB NO. 4204

SEC. NO.

SHEET NO. A-31

Forces (ref. sh. A-27) ✓

$$P_1 = 0.848 \quad \checkmark$$

$$P_2 = -0.8x \quad \checkmark$$

$$P_3 = 1.442(z-x) \quad \checkmark$$

$$\Sigma F = P_1 + P_3 - P_2 = 0 \quad \checkmark$$

$$0.848 + 1.442z - 1.442x - 0.8x = 0$$

$$2.242x = 0.848 + 1.442z \quad \checkmark$$

$$x = 0.378 + 0.643z \quad \checkmark \quad z^2 = 1.143 + .486z + .413z^2$$

$$\text{Moments @ E. 379} \quad \checkmark$$

$$M_1 = -2.111 \quad \checkmark$$

$$M_2 = -P_2 \left(\frac{x}{2}\right) = 0.8x \left(\frac{x}{2}\right) = 0.4x^2 \quad \checkmark$$

$$M_3 = 1.442z \left(\frac{z}{2}\right) + 1.442x \left(\frac{x}{2}\right) = 0.721z^2 + .721x^2 \quad \checkmark$$

$$\Sigma M = 0 \quad \checkmark$$

$$-M_1 - M_2 + M_3 = 0 \quad \checkmark$$

$$-2.111 - 0.4x^2 + 0.721z^2 + .721x^2 = 0 \quad \checkmark$$

$$-2.111 - 1.121x^2 + 0.721z^2 = 0 \quad \checkmark$$

$$-2.111 - .160 - .545z - .463z^2 + .721z^2 = 0 \quad \checkmark$$

$$-2.271 - .545z + .258z^2 = 0 \quad \checkmark$$

$$z^2 - 2.11z - 8.80 = 0 \quad \checkmark$$

$$z = \frac{2.11 \pm (4.45 + 35.2 = 39.65)^{1/2}}{2} = \frac{2.11 \pm 6.30}{2} = 8.41 \quad \checkmark$$

$$= 4.21' \quad \checkmark$$

depth of clay below E1.379 = 4' ± ✓

Say OK

Since it is possible that a differential water head could occur from the back to the front of the sheeting increase depth of embedment to 7' ✓

Use HP-115 (PMA22) total length = 7' + 7' = 14'

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# HNTB

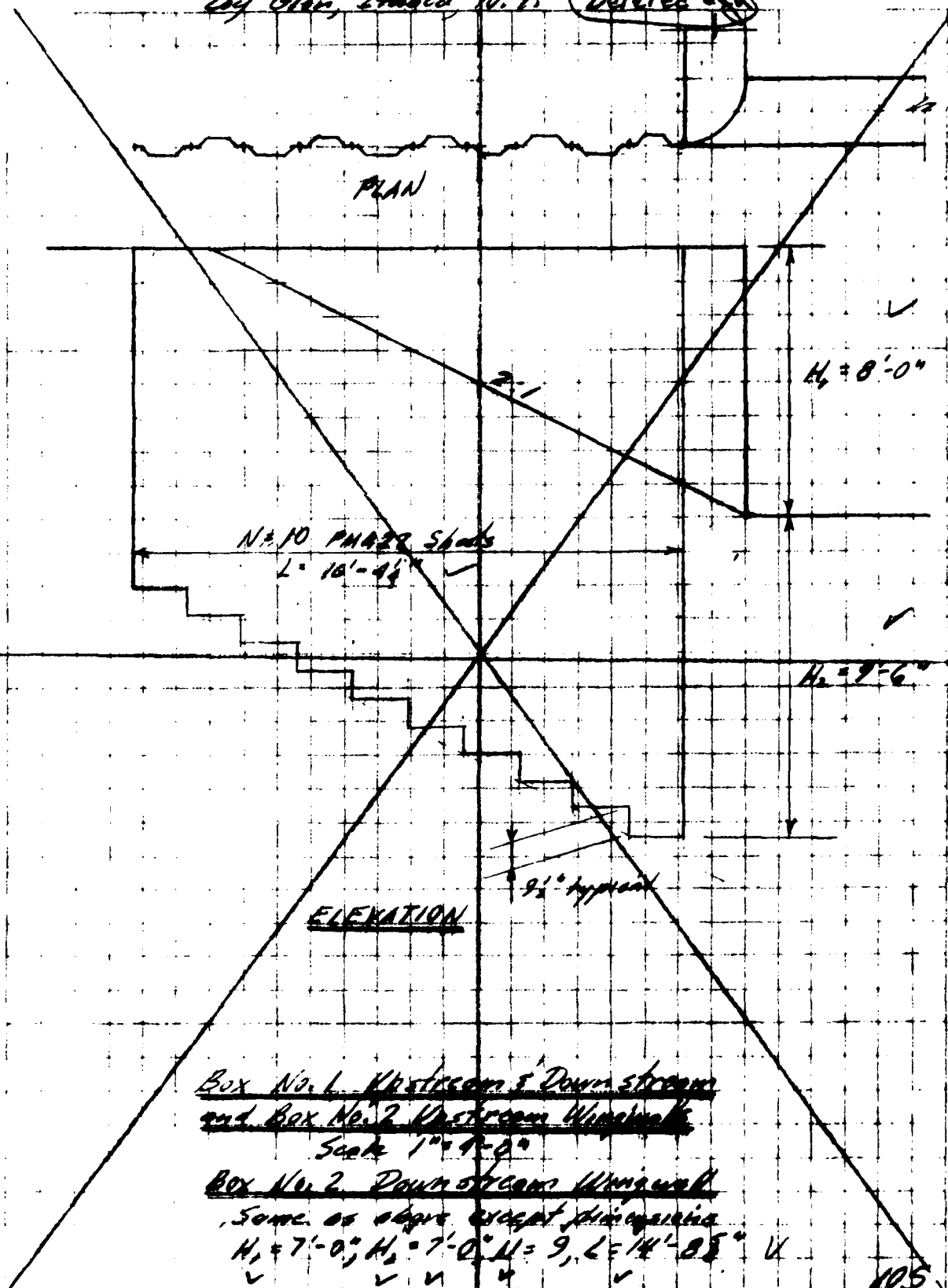
CALCULATIONS FOR

Lay Glen, Ithaca, N. Y.

Deleted

MADE BY AAS-T DATE 6-26-75 JOB NO. 1204  
 CHECKED BY J.H.T. DATE 6-30-75 SEC. NO.   
 SHEET NO. A-32

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Subject Cay Glen, Ithaca, New York

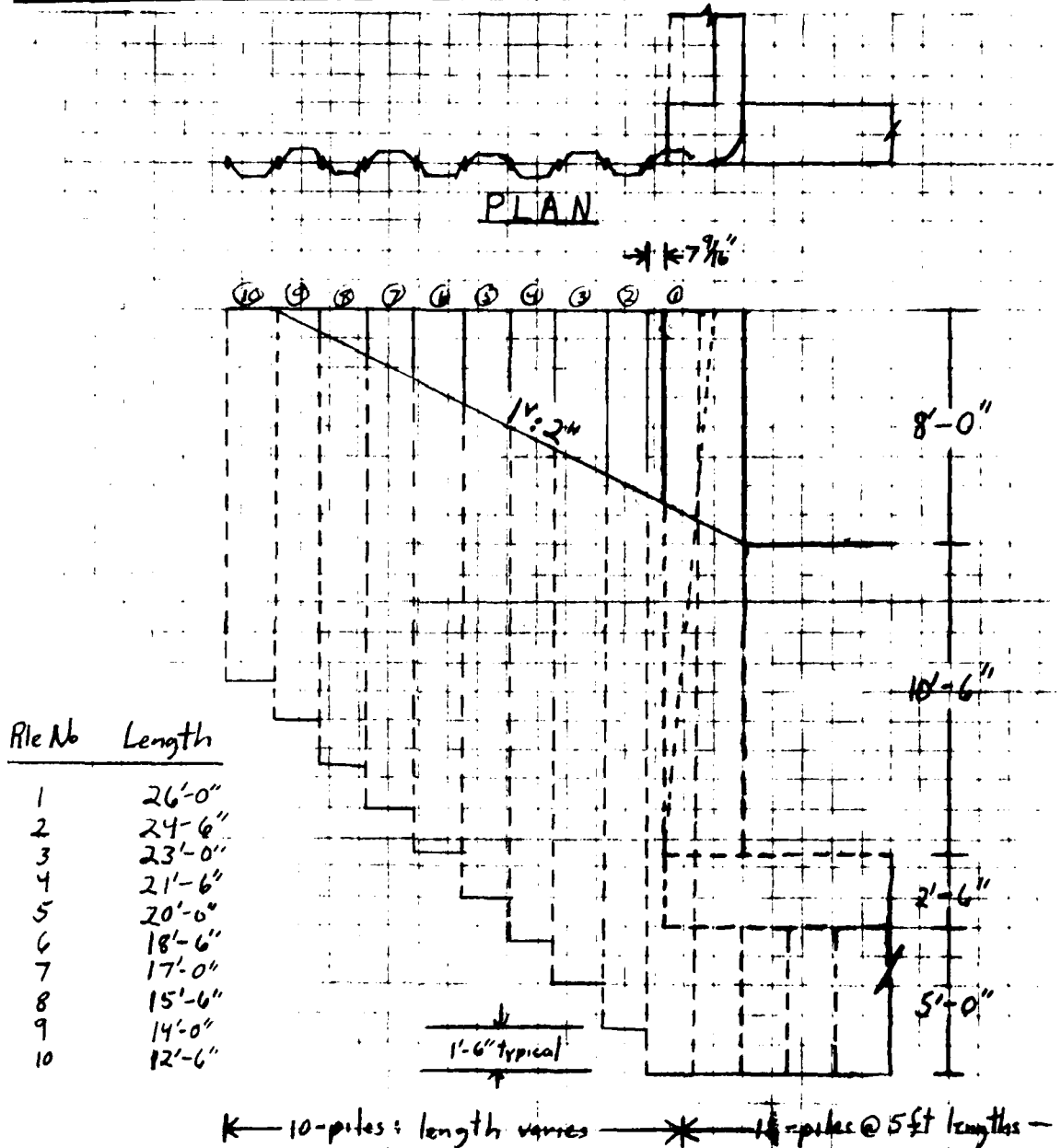
Page      of      pages  
Sheet A-32A

Computation of Sheet Pile Wingwalls at Upstream End of Drop Structures 1 & 2

Computed by RJG

Checked by ada

Date 18 Dec 1975



### ELEVATION

UPSTREAM WINGWALLS FOR

BOX NO. 1 and 2

scale:  $3/16" = 1'-0"$

105A

Subject Coy Glen, Ithaca, New York

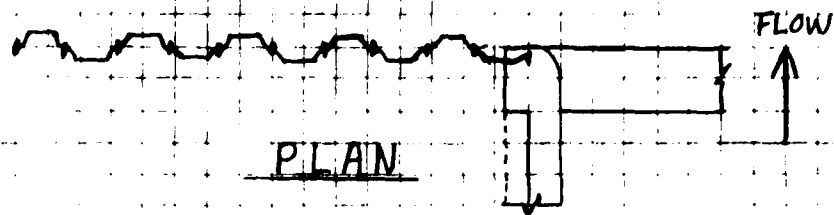
Page      of      pages.  
sheet A-32B

Computation of Sheet Pile Wingwalls at Downstream End of Pipe Structures 1 & 2

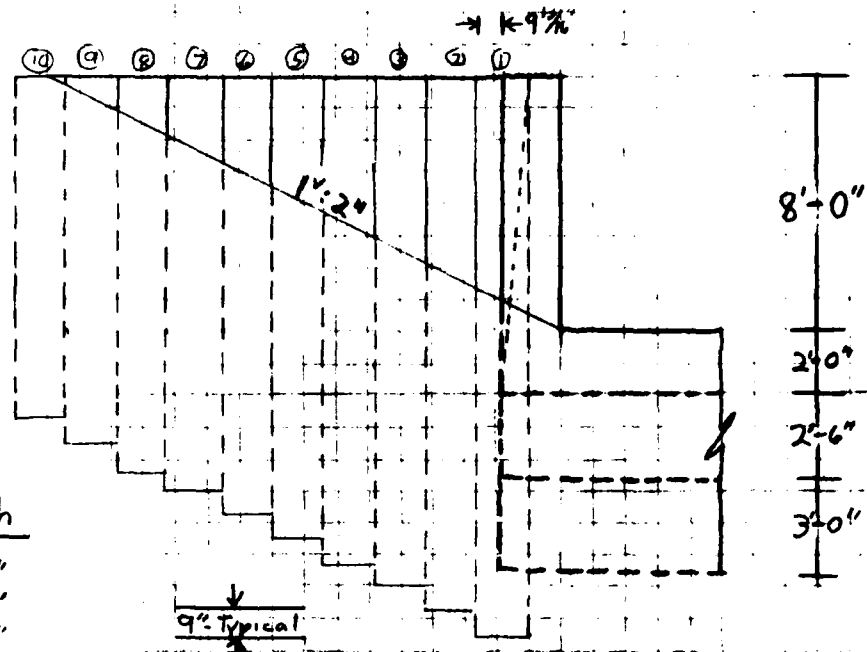
Computed by RJG

Checked by aga

Date 18 Dec 1975



PLAN



Pile No. Length

1	17'-6"
2	16'-9"
3	16'-0"
4	15'-3"
5	14'-6"
6	13'-9"
7	13'-0"
8	12'-3"
9	11'-6"
10	10'-9"

← 10-piles: length varies →

ELEVATION

DOWNSTREAM WINGWALLS

FOR

BOX NO. 1 and 2

scale:  $\frac{3}{16}'' = 1'-0''$

105 B

### 3. RIPRAP ANALYSIS

3.1 The riprap analysis is for replacement of riprap that has been eroded away in the Cayuga Inlet channel between the Lehigh Railroad bridge and the drop structure.

3.2 An inspection of the site showed that the riprap that could be observed, that on the banks of the channel, was in good condition and exhibited no signs of erosion. Riprap on the bottom of the inlet channel could not be observed because of the depth of water.

3.3 Theoretical analyses of riprap requirements were made of the inlet channel below the railroad bridge, Sheets R-1 to R-6. The theoretical analysis was in good agreement with that specified for the construction, Sheet R-6, Typical measurements of stone on the channel bank, Sheet R-10, showed that the stone placed was reasonably close to that specified for the construction.

3.4 The agreement between the above noted theoretical analysis and performance of the stone on the bank indicate the riprap performance along the channel bank is in agreement with the theoretical design at this site.

3.5 In June 1964, sieve analyses were performed on the foundation material in the area of the proposed riprap repair. An average gradation curve based on these analyses was plotted on ENG Form 2087 (see page 107A). The range for a filter design based on this foundation material and the range for the bedding material specified in the 1967 contract are also

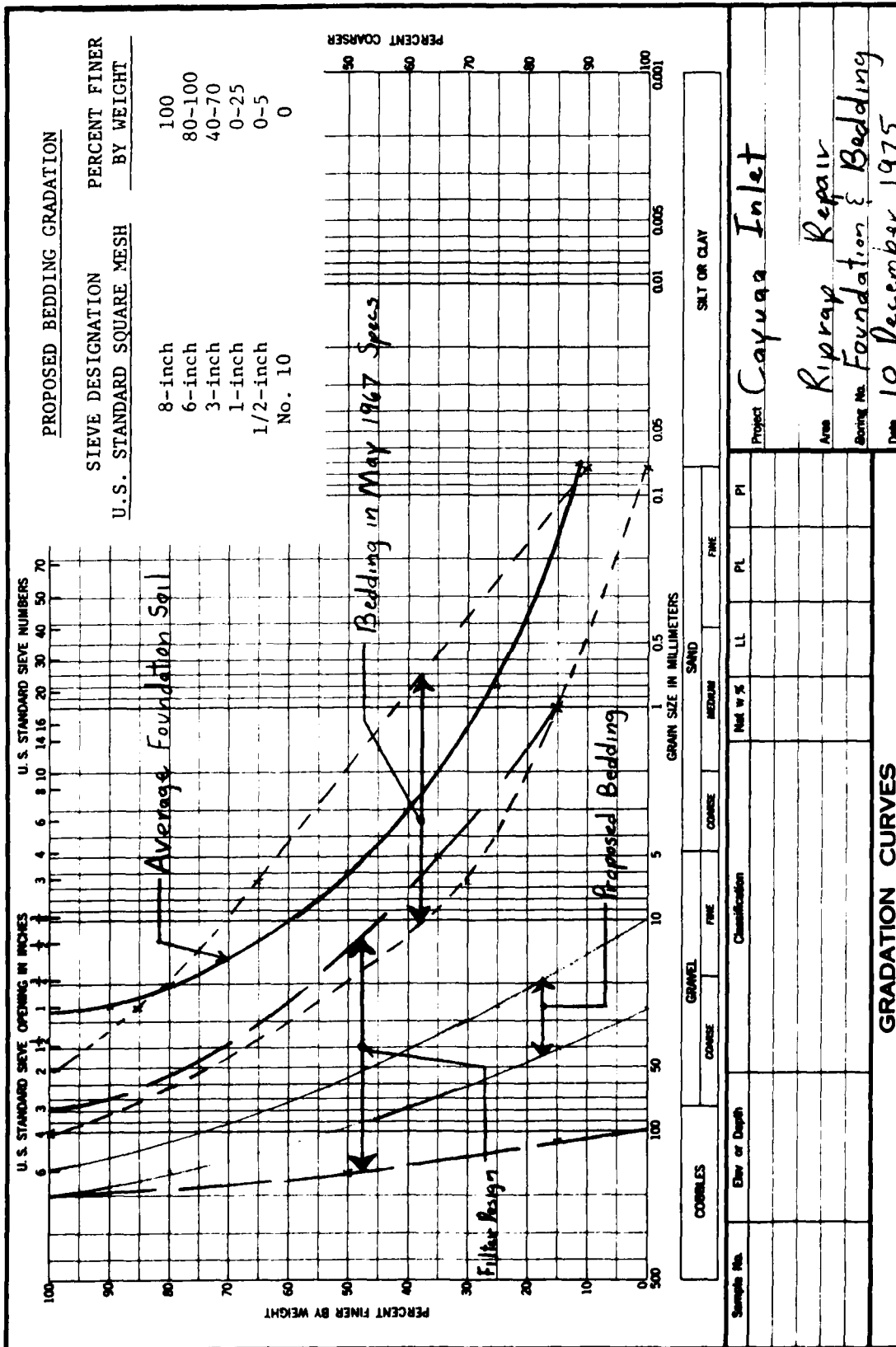
shown on ENG Form 2087. Comparison of the curves indicate that the finer sizes of the bedding gradation specified in the 1967 contract were similar to the native material and the coarser range fell within the filter design limits. Therefore, the existing foundation material and the specified bedding material in the 1967 contract are compatible. ENG Form 2087 also indicates a bedding material which falls within the range of the filter design and is the proposed bedding material to be used in the riprap repair.

3.6 A theoretical analysis was made for the riprap in the scour area just below the drop structure, Sheets R-7 and 8. It was found that the required size was essentially the same as that used on the original construction. It is concluded that the riprap failure at this location is therefore due to local turbulence occurring because of the drop structure. It appears that the drop structure is of inadequate length for full attenuation of turbulence caused by the water fall.

3.7 Stone sizes for traction shear forces up to 2.8 times the normal value were investigated on Sheet R-9. Since the stone size is a function of the third power of the traction shear force, the resulting stone sizes grew rapidly as the design traction force was increased.

3.8 The stone size proposed by the Buffalo District is about 2.5 times the size theoretically required if turbulence were not present. It also results in an increased traction shear resistance of 25 percent. The use a larger size stone to provide 50 percent increase in traction





ENG FORM 2087  
1 MAY 63

shear resistance results in a stone size 4 times that theoretically required. The use of any larger size stone is not practical since it would be larger than the scour hole it is to fill.

3.9 It is concluded that the size stone to be used in the scour area will have to be based on judgment since no evaluation of the turbulence present in this area under high flow can be made. The size riprap proposed by the Buffalo District, Sheets R-11 to R-13 are reasonable since it is two and a half times larger than that which was eroded out.

3.10 Riprap designs for the two adjacent Coy Glen drop structures developed by the Buffalo District are included on Sheets R-14 to R-18.

# HNTB

CALCULATIONS FOR

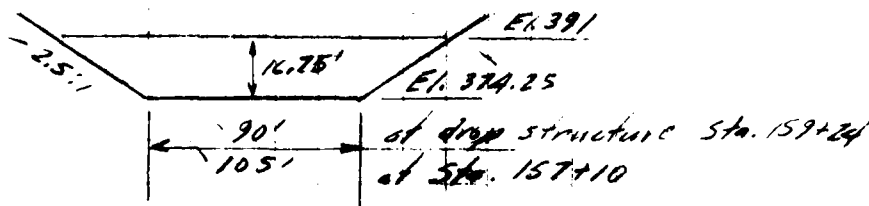
Coy Glen, Ithaca, N.Y.

MADE BY AAS-J DATE 4-7-75 JOB NO. 420A-99-01  
 CHECKED BY COV DATE April 11-75 SEC. NO.   
 SHEET NO. R-1

## Coyuga Inlet Rip-Rap Design Sta. 157+10 to 159+24

Design Flow 16,000 cfs Ref. Inc. F-2  
 Bottom of Channel El. 374.25 " " F-3, Dwg 238-A-31/5  
 Top of Water El. 391 " Phone call 4-7-75 to Mr.  
 R. Gorecki DD, C of E  
 Av. velocity 7.1 to 7.7 ft/sec. " - do -

## Channel Section Ref. Inc. F-3 (Dwg. 238-A-31/5)



$$\begin{aligned} \text{Av. Width } W_1 &= 90 + (2.5 \times 16.75) = 91.875' \\ W_2 &= 105 + 16.875 = 146.875' \end{aligned}$$

$$\begin{aligned} \text{Area } A_1 &= 16.75 \times 91.875 = 2,208.9 \text{ s.f.} \\ A_2 &= 16.75 \times 146.875 = 2,460.2 \text{ s.f.} \end{aligned}$$

$$\begin{aligned} \text{av. Velocity } V_1 &= 16,000 \div 2,208.9 = 7.24 \text{ ft/sec. vs 7.7} \\ V_2 &= 16,000 \div 2,460.2 = 6.50 \text{ ft/sec. vs 7.1} \end{aligned}$$

Note: Inc. F-3 shows top of Rip Rap @ El. 390.0

For top of water at El. 390.0

$$\begin{aligned} \text{Av. Width } W_1 &= 90 + (2.5 \times 15.75) = 129.375' \\ A_1 &= 15.75 \times 129.375 = 2,037.7 \text{ s.f.} \\ V_1 &= 16,000 \div 2,037.7 = 7.85 \text{ ft/sec. vs 7.7} \\ W_2 &= 105 + 15.75 = 144.375' \\ A_2 &= 15.75 \times 144.375 = 2,273.9 \text{ s.f.} \\ V_2 &= 16,000 \div 2,273.9 = 7.0 \text{ ft/sec. vs 7.1} \end{aligned}$$

USE av. velocity = 7.7 ft/sec.  
 depth = 15.75' 108

**HNTB**

CALCULATIONS FOR

Coy Glen, Ithaca, N.Y.

MADE BY ADD-T DATE 9-8-15 JOB NO. 9204  
CHECKED BY COV DATE April 11-75 SEC. NO.   
SHEET NO. R-2Local Boundary Shear Ref EM 1110-2-1601, Eq. 32

$$\tau_0 = \gamma \left[ \frac{V}{32.6 \log_{10} \frac{12.24}{D_{50}}} \right]^2$$

$V = \text{velocity} = 7.7 \text{ fps}$  (more critical than  $V = 7.1$ )

$y = \text{channel depth} = 15.75'$

$D_{50} = \text{av. stone diameter} = \text{try } 10'$

$\gamma = \text{unit wt. of water} = 62.4 \text{ pcf}$

$$\frac{12.24}{D_{50}} = \frac{12.2 \times 15.75}{10} = 192; \log_{10} \frac{12.24}{D_{50}} = 2.283$$

$$32.6 \times \log_{10} ( ) = 32.6 \times 2.283 = 74.3$$

$$V \div 32.6 \log ( ) = 7.7 \div 74.3 = 0.1035$$

$$[V \div 32.6 ( )]^2 = 0.1035^2 = 0.0107$$

$$\tau_0 = 62.4 [ ]^2 = 62.4 \times 0.0107 = 0.67 \text{ pcf}$$

$$\tau_{0\text{max}} / \tau_0 = 2.0 \text{ for smooth channel EM 1110-2-1601 PL33}$$

$$= 2.8 \text{ " rough "}$$

$$\text{USE } \tau_{0\text{max}} / \tau_0 = 2.8$$

$$\tau_{0\text{max}} = 2.8 \times 0.67 = 1.88 \text{ pcf}$$

$$\text{Design } \tau_0 = 1.5 \tau_{0\text{max}} = 1.5 \times 1.88 = 2.82 \text{ pcf Ref. ETL 1110-2-120 3.0(4)}$$

For side slope of (Vern. 2.5H channel)

$$\tau' = \tau \left( 1 - \frac{\sin^2 \phi}{\sin^2 \theta} \right)^{\frac{1}{2}} \quad \text{Ref EM 1110-2-1601 Eq. 34}$$

$$\phi = \tan^{-1} 2.5 = \tan^{-1} 0.4 = 21.8^\circ \quad \sin 21.8^\circ = 0.371$$

$$\sin^2 21.8^\circ = 0.138$$

$$\theta = 40^\circ \quad \sin 40^\circ = 0.642, \quad \sin^2 40^\circ = 0.413$$

$$\tau' = 2.82 \left[ 1 - \left( \frac{0.138}{0.413} = 0.335 \right) \right]^{\frac{1}{2}} = 2.82 \times 0.815 = 2.30 \text{ pcf}$$

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HOWARD NEEDLES TAMMEN &amp; BERENSON OFF

CONSULTING ENGINEERS

Coy Glen, Ithaca, N.Y.

MADE BY AASV DATE 4-8-75 JOB NO. 4204  
 CHECKED BY COV DATE Apr 11-75 SEC. NO. R-3  
 SHEET NO. R-3

Req'd Shear Resistance of Rip-Rap  
Channel bottom  $\tau = 2.82$   
" side  $\tau' = 2.30$

$$D_{50} = \frac{\tau}{a(\tau_s - \tau)} \quad \text{Ref. EM 1110-2-1601 Eq. 33}$$

$$a = 0.040$$

$\tau_s =$  unit wt. of stone

Limestone  $\tau_s = 152$  to  $166$

Sandstone  $119$  to  $161$

USE  $\tau_s = 160$  pcf

$\tau = 02.4$

Ref: Table 2.1, Kyriak  
 & Todd "Principles of  
 Engineering and  
 Geotechnical Engineering"

$$\text{Req'd min. } D_{50} = \frac{2.82}{0.04(160 - 02.4)} = \frac{2.82}{(0.04 \times 157.6 = 6.30)} = 0.72'$$

$$\text{Req'd } V = \frac{\pi D^3}{6} = \frac{\pi (0.72)^3}{6} = \frac{\pi}{6} 0.373 = 0.195 \text{ cf.}$$

$$\text{Req'd min. } W_{50} = 0.195 \times 160 = 31.2 \text{ lbs.}$$

For  $\tau_s = 160$  pcf, Thicknesses =  $24''$  min  $W_{50} = 40$  lbs.

Ref. ETL 1110-2-120

A Check for  $\tau_s = 155$  pcf

Incl 2, p 2 \*

$$\text{Req'd min } D_{50} = \frac{2.82}{0.04(155 - 02.4 = 152.6)} = 0.76'$$

$$\text{Req'd } V = \frac{\pi}{6} (0.76^3 = 0.44) = 0.230 \text{ cf.}$$

$$\text{Req'd min. } W_{50} = 0.23 \times 155 = 35.6 \text{ lbs.}$$

Note for  $\tau = 155$  15" Thickness min  $D_{50} = 38''$  (Ref ETL 1110-  
 18" 55" 2-120, Incl 2, p 2) \*

Check  $\tau_s$  for min.  $D_{50} = 40''$   $\tau = 160$  pcf  
 = 38" of 55" for  $\tau = 155$  pcf

\* Assumed to be placed under water.

Coy Glen, Ithaca, N.Y.

(1) For min.  $D_{50} = 40\#$ ,  $\gamma = 160 \text{ pcf}$

Design shear Force,  $T_b \text{ max.}$

$$Vol. = 40 \div 160 = 0.250 \text{ ft}^3$$

$$D = \left( \frac{6V}{\pi} \right)^{1/3} = \left( \frac{6 \times 0.250}{\pi} = 0.477 \right)^{1/3} = 0.781'$$

calc.  $T_b$

$$\frac{12.24}{D_{50}} = \frac{12.24 \times 15.75}{0.781} = 246$$

$$\log_{10} \frac{12.24}{D_{50}} = 2.390$$

$$32.6 \log_{10} ( ) = 32.6 \times 2.390 = 78.0$$

$$V \div 32.6 \log_{10} ( ) = 7.7 \div 78.0 = 0.0986$$

$$[V \div 32.6 \log_{10} ( )]^2 = 0.0986^2 = 0.00970$$

$$T_b = 62.4 \times 0.0097 = 0.606 \text{ psf}$$

$$\text{Design } T_b \text{ max} = 0.606 \times 2.8 \times 1.5 = 2.54 \text{ psf}$$

design  $T_b \text{ max} = 1.5 T_b \text{ max}$  (SL-R2)  
 for rough channel  $T_b \text{ max} / T_b = 2.8$  (SL-R2)

Inplace shear resistance,  $T_b$  &  $T_s$

$$\begin{aligned} \text{(a) bottom: } T &= a(\gamma_s - \gamma) D_{50} = 0.04(160 - 62.4 \times 92.6) 0.781 \\ &= 3.05 > 2.54 \text{ channel bottom OK} \end{aligned}$$

(b) channel side

$$\left( 1 - \frac{\sin^2 \phi}{\sin^2 \theta} \right)^{1/2} = 0.815 \text{ (SL-R-2)}$$

$$T' = 3.05 \times 0.815 = 2.48 \approx 2.54 \text{ channel sides - Say OK}$$

Conclusion: min  $D_{50} = 40\#$ ,  $\gamma = 160 \text{ pcf OK}$

$$2.48 \div 2.54 = 0.98 \text{ 2\% low}$$

(2) For min.  $D_{50} = 38\#$ ,  $\gamma = 155 \text{ pcf}$

Design Shear Force  $T_b \text{ max.}$

$$Vol = 38 \div 155 = 0.245 \text{ ft}^3 \text{ from (1) above then}$$

$$D = 0.777' \text{ and design } T_b \text{ max} = 2.54$$

Inplace Shear Resistance;  $T_b$  &  $T_s$

$$\text{(a) bottom: } T = 0.040(155 - 62.4 \times 92.6) 0.777 = 2.88 > 2.54 \text{ OK}$$

$$\text{(b) side } T' = 2.88 \times 0.815 = 2.35 < 2.54 \text{ NG.}$$

Conclusion: min.  $D_{50} = 32\#$ ,  $\gamma = 155 \text{ pcf N.G.}$

$$2.35 \div 2.54 = 0.93 \text{ 7\% low. 111}$$

**HNTB**

CALCULATIONS FOR

MADE BY AAS-J DATE 8-8-75 JOB NO. 4206  
 CHECKED BY CV DATE April 11-75 SEC. NO. R-5  
 SHEET NO. R-5

Coy Glen, Ithaca, N.Y.

- 3) For min.  $D_{50} = 55/16$ ,  $T = 155$  pcf  
Design Shear Force,  $T_{6max}$   
 $Vol. = 55 \div 155 = 0.354$  c.f.

$$D = \left( \frac{6 \times 0.354}{\pi} \right)^{1/3} = 0.677^{1/3} = 0.877'$$

$$\text{Calc. } T_0 = \frac{12.24}{D_{50}} = \frac{12.2 \times 15.75}{0.877} = 219$$

$$\log_{10} \frac{12.24}{D_{50}} = 2.34$$

$$32.6 \log_{10} ( ) = 32.6 \times 2.34 = 76.2$$

$$V \div 32.6 \log_{10} ( ) = 7.7 \div 76.2 = 0.101$$

$$[V \div 32.6 \log_{10} ( )]^2 = 0.101^2 = 0.0102$$

$$T_0 = 62.4 \times 0.0102 = 0.64 \text{ pcf}$$

$$\text{Design } T_{6max} = 0.64 \times 2.8 \times 1.5 = 2.69 \text{ pcf}$$

Inplace Shear Resistance,  $T$  &  $T'$

$$(a) \text{ bottom } T = a(T_0 - T) D_{50} = 0.000(155 - 62.4 = 92.6) 0.877$$

$$= 2.25 \text{ pcf} > 2.69 \text{ OK}$$

$$(b) \text{ s.s. } T' = 3.25 \times 0.815 = 2.65 \approx 2.69 \text{ say OK}$$

Ref. ETL 1110-2-120 3.C.(5)  
 $2.65 \div 2.69 = 0.98$  2% low

- (4) Check for  $T = 150$  pcf per R. Goretzki by phone 7 Apr. 75

Try min  $D_{50} = 73/16$  Ref. ETL 1110-2-120 Encl. 2, p1

Design Shear Force,  $T_{6max}$

$$Vol. = 73 \div 150 = 0.487 \text{ c.f.}$$

$$D = \left( \frac{6 \times 0.487}{\pi} \right)^{1/3} = 0.93^{1/3} = 0.976'$$

$$\text{Calc. } T_0 = \frac{12.24}{D_{50}} = \frac{12.2 \times 15.75}{0.976} = 197$$

$$\log_{10} \frac{12.24}{D_{50}} = 2.294$$

$$32.6 \log_{10} ( ) = 32.6 \times 2.294 = 75.0$$

$$V \div 32.6 \log_{10} ( ) = 7.7 \div 75 = 0.1025$$

$$[V \div 32.6 \log_{10} ( )]^2 = 0.0105$$

$$T_0 = 62.4 \times 0.0105 = 0.655 \text{ pcf}$$

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HOWARD NEEDLES TAMMEN &amp; BERENSON OFF CONSULTING ENGINEERS

# HNTB

CALCULATIONS FOR

Coy Glen, Ithaca, N.Y.

MADE BY AAS-T DATE 4-8-75 JOB NO. 4204  
 CHECKED BY CUU DATE April 11-75 SEC. NO.   
 SHEET NO. R-6

$$\text{Design } T_{b \max} = 0.655 \times 2.8 \times 45 = 2.75 \text{ psf}$$

Inplace Shear Resistance  $T' \text{ of } 2'$

$$(a) \text{ bottom } T' = a(T_b - \gamma) P_{sa} = 0.04(150 - 62.4 = 87.6) 0.976$$

$$= 3.94 \text{ psf} \quad 2.75 \text{ OK}$$

$$(b) \text{ side } T' = 3.42 \times 0.845 = 2.79 \text{ psf} \quad 2.75 \text{ OK}$$

$$2.75 - 2.79 = 1.98 \text{ 2\% low}$$

## Summary

1) for unit wt. of stone,  $\gamma_s = 160 \text{ pcf}$   
 min.  $D_{50} = 40/165$  Ref. Sh. R-4  
 Percent Passing Stone Size Ref. ETL 1110-2-120  
 100 199-79 Incl. 2, p 2  
 50 59-40  
 15 29-12

3) for unit wt. of stone,  $\gamma_s = 155 \text{ pcf}$  Ref. same  
 min.  $D_{50} = 55/165$  Ref. Sh. R-5  
 100 274-110  
 50 81-55  
 15 41-17

4) for unit wt. of stone,  $\gamma_s = 150 \text{ pcf}$  Ref. same  
 min.  $D_{50} = 73 \text{ pcf}$  Ref. Sh. R-6 p1  
 100 364-145  
 50 108-73  
 15 54-23

Note: Corps of Engineer Specs for Existing Cayuga Inlet  
 Construction Specs for Rip-Rap (Mr. R. Gorecki  
 by phone May 2, 1975)

$D_{100}$  280 lb.  
 $D_{50}$  65 lb.  
 $D_{15}$  15 lb.

HOWARD NEEDLES TAMMEN & BERGENDOFF CONSULTING ENGINEERS



**HNTB**

CALCULATIONS FOR

Cox Glen, Ithaca, N.Y.

 MADE BY AAS-J DATE 4-21-75 JOB NO. 4204-99-01  
 CHECKED BY SWE DATE 5/7/75 SEC. NO. R-7  
 SHEET NO. R-7
Channel Section @ Sta. 160+00Check velocity

Width 80' (Ref. Enc. F-7)

Bottom Elev 374.28 (Ref. F-3, Dig 238-A-31/5)

Water surface El. 390 (top of rip-rap Enc. F-3)

$$y = 15.72'$$

$$\text{Area} = (390 - 374.28 \times 15.72) 80 = 1257.6 \text{ sq ft}$$

Design Flow 16,000 cfs (Enc. F-2)

$$\text{Av. Vel.} = 16,000 \div 1257.6 = 12.7 \text{ ft/sec.}$$

Local Boundary Shear Ref. EM 1110-2-1601, Eq. 3cAssume  $D_{50} = 1.0$ 

$$T_0 = \tau \left( 32.6 \log_{10} \frac{12.24}{D_{50}} \right)^2$$

$$\frac{12.24}{D_{50}} = \frac{12.2 \times 15.7}{1} = 191.5; \log_{10} \frac{12.24}{D_{50}} = 2.282$$

$$32.6 \log_{10} [ ] = 32.6 \times 2.282 = 74.4$$

$$T_0 = 62.4 \left( \frac{12.7}{74.4} = 0.171 \right)^2 = 62.4 \times 0.03 = 1.87$$

1) Design  $T_0$  use 1.5  $T_0$  Ref. ETL 1110-2-120 Sec. 3.d(4)

$$= 1.5 \times 1.87 = 2.80 \text{ psf vs } 2.82 \text{ ab. R-2}$$

\* based on design  $T_0 = 1.5 \times 2.8 T_0$  where 2.8 factor is for conditions of bend in channel.
 2) Since turbulence is likely to exist at this location and it could be on the order of that encountered at a bend (Ref. R 33, EM 1110-2-1601) also check Design  $T_0 = 2.8 T_0 = 2.8 \times 2.8 = 7.84 \text{ psf}$ 

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**HNTB**

CALCULATIONS FOR

Coy Glen Shale, N.Y.

MADE BY NAS-J DATE 8-21-75 JOB NO. 4204  
 CHECKED BY SWC DATE 5/7/75 SEC. NO.   
 SHEET NO. R-8

For  $\gamma = 150 \text{ pcf}$ 1) For design  $T_o = 2.8 \text{ pcf}$ 

$$\text{Req'd } D_o = \frac{\gamma}{a(\gamma_s - \gamma)} = \frac{2.8}{0.04(150 - 62.4)} = \frac{2.8}{0.04 \times 87.6} = 0.8'$$

$$\text{Req'd volume} = \frac{\pi D^3}{6} = 0.27 \text{ cf; req'd wt} = 0.27 \times 150 = 40^{\#}$$

From ETL 110-2-120, Table 2, page 1 for  $\gamma = 150 \text{ pcf}$   
 Min.  $D_{50} = 63 \text{ pcf}$

$$V_o = 53 \div 150 = 0.354 \text{ c.f.}$$

$$\text{req'd } D^3 = \frac{6V}{\pi} = \frac{6 \times 0.354}{\pi} = 0.676$$

$$D = (0.676)^{1/3} = 0.878 \text{ ft.}$$

Design Shear,  $T_o'$ , for  $D = 0.88'$ 

$$\frac{12.24}{D_{50}} = \frac{12.2 \times 15.72}{0.88} = 218'; \log_{10} \frac{12.24}{D_{50}} = 2.338$$

$$32.6 \log_{10} \frac{12.24}{D_{50}} = 32.6 \times 2.338 = 76.3$$

$$T_o = 62.4 \left( \frac{12.7}{76.3} = 0.166 \right)^{1/2} = 62.4 \times 0.278 = 1.73 \text{ pcf}$$

$$\text{Design } T_o' = 1.5 T_o = 1.5 \times 1.73 = 2.59 \text{ pcf}$$

$$\begin{aligned} \text{Shear Resistance; } \bar{E} &= D_o a (\gamma - \gamma_s) \\ &= 0.88 \times 0.04 (150 - 62.4 = 87.6) \\ &= 3.08 \text{ pcf} > 2.59 \text{ OK} \end{aligned}$$

Min  $D_{50} = 53^{\#}$ From Sh. R-6 existing  $D_o = 6.5 \text{ ft} > 53 \text{ ft}$ .

Since existing rip-rap, which has eroded, is essentially equal to theoretical, turbulence must exist on a shear force  $T_o > 2.8$  should be checked.

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# HNTB

CALCULATIONS FOR

Coy Glen, Ithaca, N.Y.

MADE BY AAZ-Y DATE 5-6-75 JOB NO. 4604  
 CHECKED BY SWE DATE 5/7/75 SEC. NO.   
 SHEET NO. R-9

Check req'd  $D_{50}$  for  $T_o = 1.25, 1.5, 2.0$  &  $2.8 \times 2.8$

2) For  $T_o = 2.8 \times 2.8 = 7.84$  psf

$$\text{Req'd } D_{50} = \frac{T_o}{0.04 \times 87.6} = \frac{7.84}{3.504} = 2.24'$$

$$\text{Req'd Volume} = \frac{\pi D^3}{6} = \frac{\pi (2.24)^3}{6} = 5.88 \text{ cf}$$

@ 150 psf,  $W_{50} = 882$  lbs

Req'd size appears too large

3) For  $T_o = 1.5 \times 2.8 = 4.2$  psf

$$\text{Req'd } D_{50} = \frac{4.2}{0.04 \times 87.6} = 1.20'$$

$$\text{Req'd Vol.} = \frac{\pi (1.20)^3}{6} = 0.90 \text{ cf @ 150 psf, } W_{50} = 135 \text{ lbs}$$

4) For  $T_o = 2.0 \times 2.8 = 5.6$  psf

$$\text{Req'd } D_{50} = \frac{5.6}{0.04 \times 87.6} = 1.60'$$

$$\text{Req'd Vol.} = \frac{\pi (1.60)^3}{6} = 2.14 \text{ c.f. @ 150 psf } W_{50} = 321 \text{ lb.}$$

5) For  $T_o = 1.25 \times 2.8 = 3.5$  psf

$$\text{Req'd } D_{50} = \frac{3.5}{0.04 \times 87.6} = 1.00'$$

$$\text{Req'd Vol.} = \frac{\pi (1.00)^3}{6} = 0.52 \text{ c.f. @ 150 psf, } W_{50} = 78 \text{ lbs.}$$

## Summary

No.	1)	5)	3)	4)	2)	Lot E
Design $T_o$	2.8	3.5	4.2	5.6	7.84	Design
min. $W_{50}$	53#	78#	135#	321#	882#	

ETL 1110-2-170 Incl. 2 Recommendations

$W_{120}$	265-106	484-194	998-399	2121-848	-	700-250
$W_{50}$	79-53	143-97	296-200	628-424	-	250-135
$W_{15}$	39-17	72-30	148-62	314-133	-	100-40
T	22"	33"	42"	54"	-	116

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# HNTB

CALCULATIONS FOR

Coy Glen, Ithaca, N.Y.

MADE BY AAS-7 DATE 5-2-17 JOB NO. 4204  
CHECKED BY \_\_\_\_\_ DATE \_\_\_\_\_ SEC. NO. \_\_\_\_\_  
SHEET NO. R-10

Observed Rip-Rap - Cyonga Channel below  
drop structure, south bank

Largest Stone

$1.0 \times 1.3 \times 1.7 = 2.2 \text{ c.f.}$	@ 150 pct	= 330 lb
$2.0 \times 1.1 \times 1.6 = 3.5$	"	525 lb
$1.4 \times 1.5 \times 1.6 = 3.35$	"	= 500 lb

Median Stone

$0.6 \times 0.6 \times 0.8 = 0.29 \text{ c.f.}$	@ 150 pct	= 44 lb
$0.6 \times 0.6 \times 0.9 = 0.32$	"	48 lb
$0.6 \times 0.7 \times 0.7 = 0.29$	"	44 lb

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HOWARD NEEDLES TAMMEN & BERGENCOFF

**RIPRAP REPAIR  
for  
CAYUGA INLET**

1. The stone for riprap shall be placed in a layer 24 inches thick and shall conform to the following gradation and as shown on Plate 1 attached to this Inclosure:

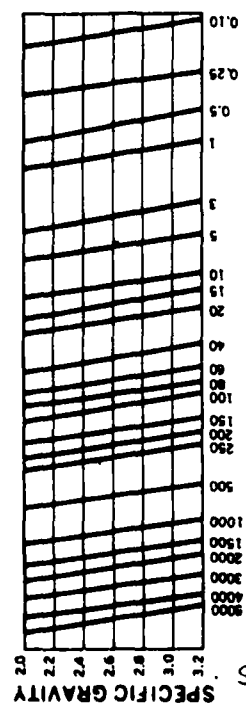
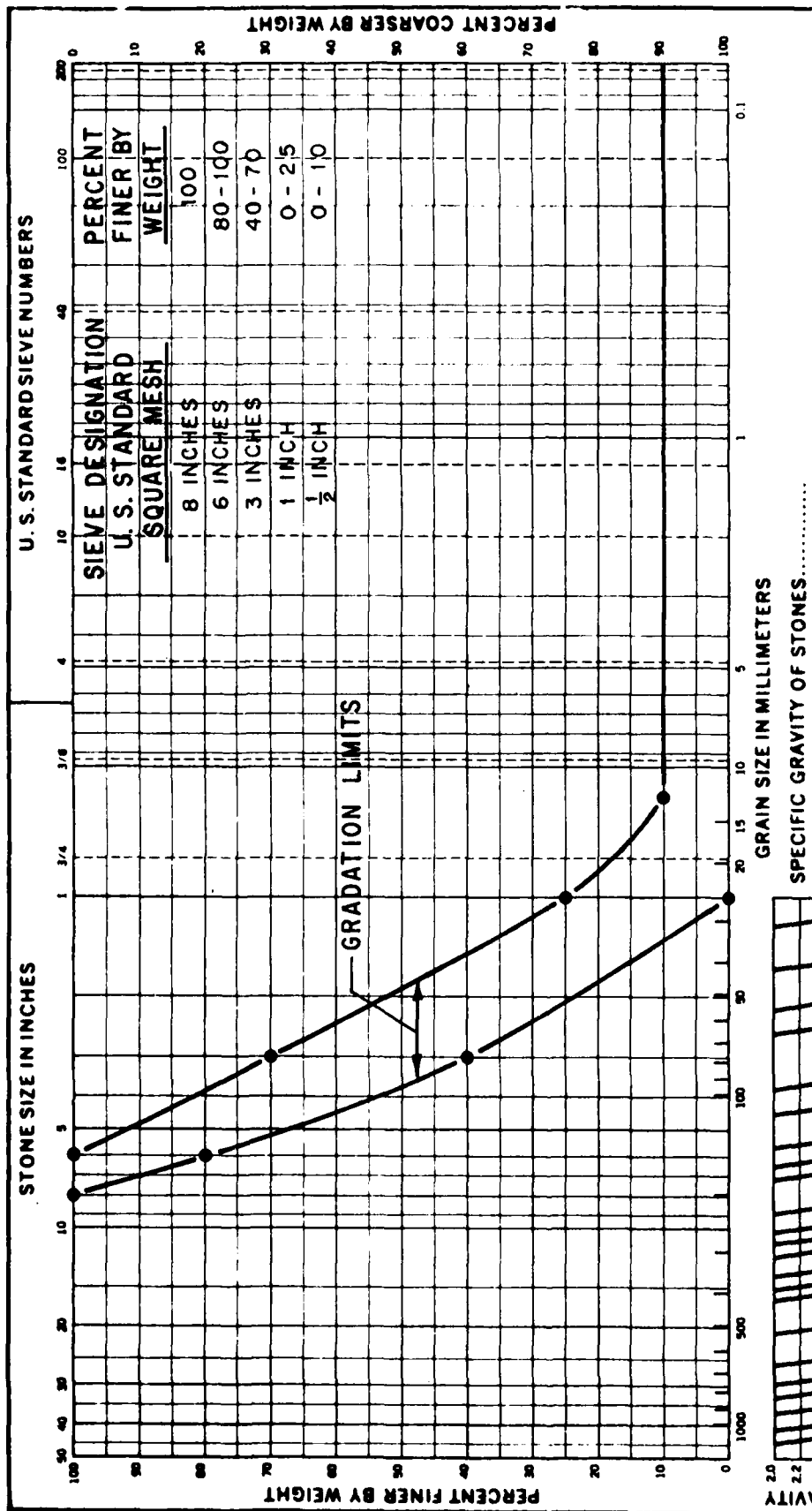
Riprap Gradation			
% Lighter by Weight	:	Limits of Stone, Weight in Pounds	
		Maximum	Minimum
100	:	700	250
50	:	250	135
15	:	100	40

2. Where sufficient material has been eroded to require underlayers, the following are recommended:

- a. Spalls - see gradation below and Plate 2 attached to this inclosure
- b. Sand and/or gravel - similar to a concrete aggregate mix

Spalls Gradation		
U.S. Standard Sieve Size (inches)	:	Percent Passing By Weight
8	:	100
6	:	80-100
3	:	40-70
1	:	0-25
1/2	:	0-10





Sh. R-13

PROJECT COY GLEN AND CAYUGA INLET  
 AREA TYPE C SPALLS  
 DATE DESIGN ANALYSIS, AUGUST 1975  
 GRADATION CURVES  
 FOR RIPRAP FILTER AND BEDDING

ENG FORM 4056  
 APR 67

RIPRAP DESIGN  
FOR  
COY GLEN

1. The riprap to be placed 25 feet upstream and downstream of each drop structure and in the channel section from stations 0+00 to station 0+25 shall be in a layer 24 inches thick with 9 inches of bedding and shall conform to the following gradations and as shown on plates 1 and 2 immediately following.

Riprap Gradation			
% by Weight Passing	:	Limits of Stone Weight in Pounds	
		Maximum	Minimum
100	:	700	250
50	:	250	135
15	:	100	40

Bedding Gradation		
U. S. Standard Sieve Size (inches)	:	Percent Finer By Weight
4	:	100
2	:	65-100
1	:	50-90
3/4	:	45-83
No. 4	:	25-60
10	:	14-48
40	:	0-30
No. 200	:	0-10

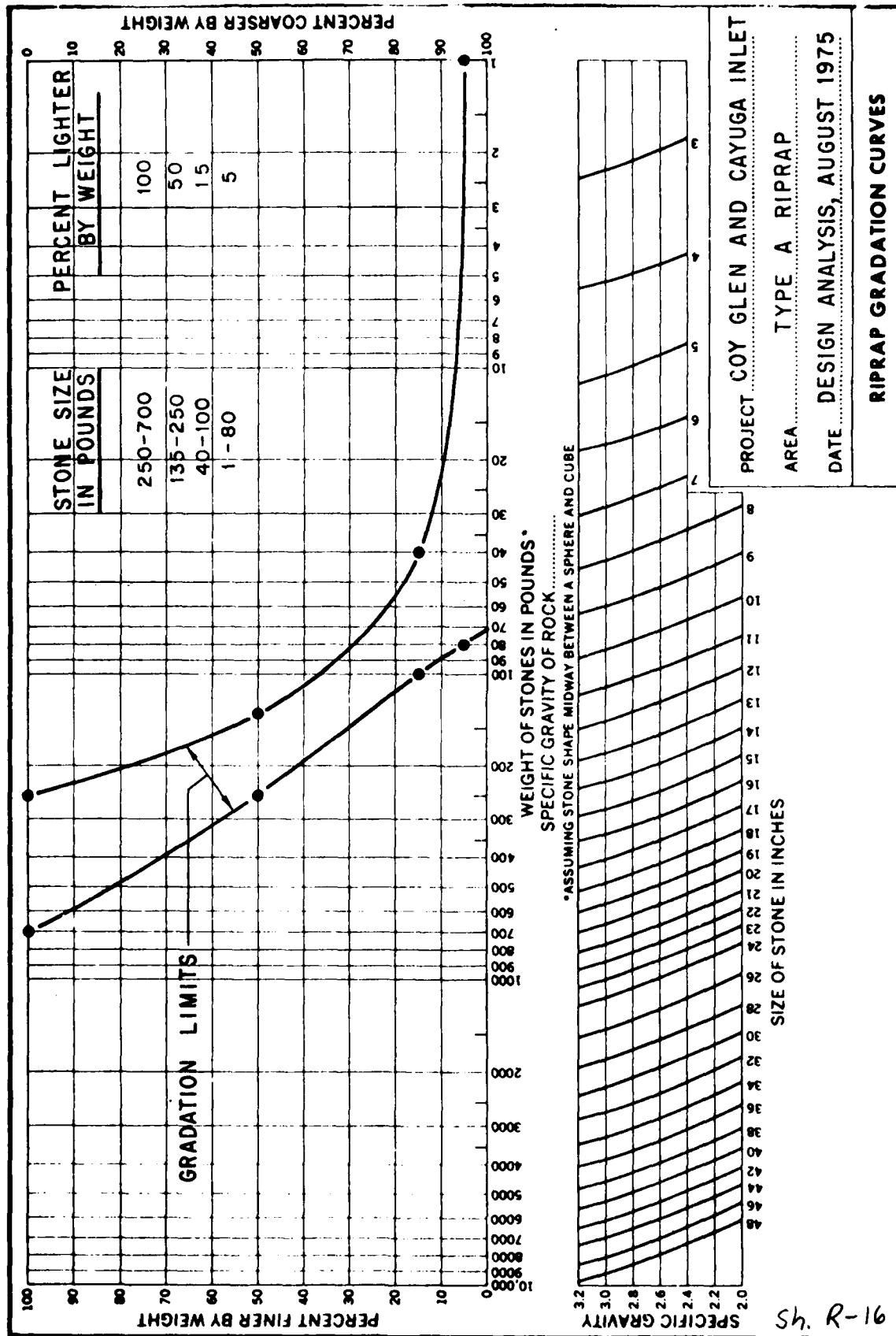
Sh. R-14



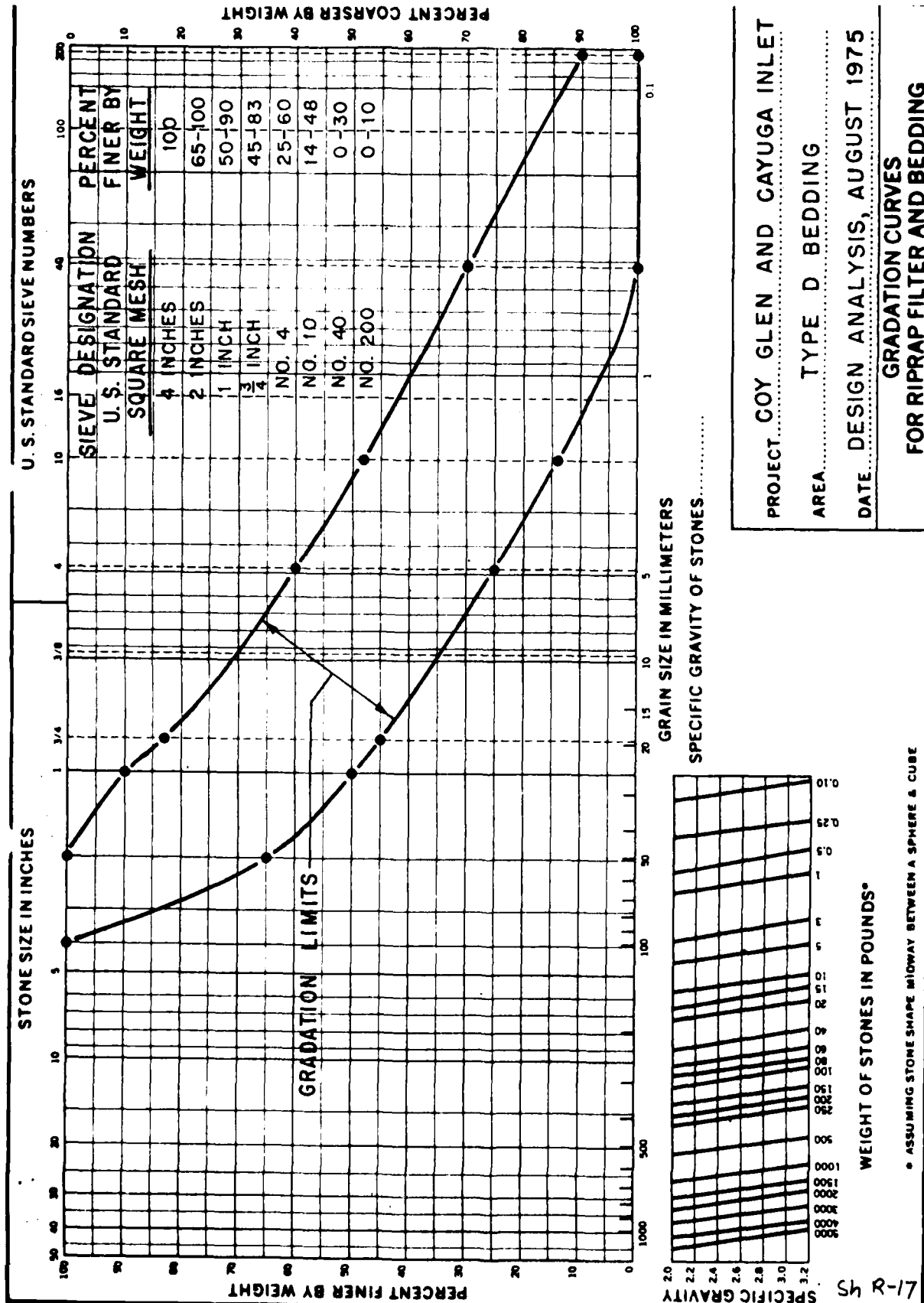
2. In the areas between station 0+25 to station 1+75 and stations 2+53 to station 3+03, the riprap shall be placed in a layer 21 inches thick with 9 inches of bedding. The riprap shall conform to the following gradation and as shown on plate 3; the bedding material shall conform to the gradation above and as shown on plate 2 immediately following.

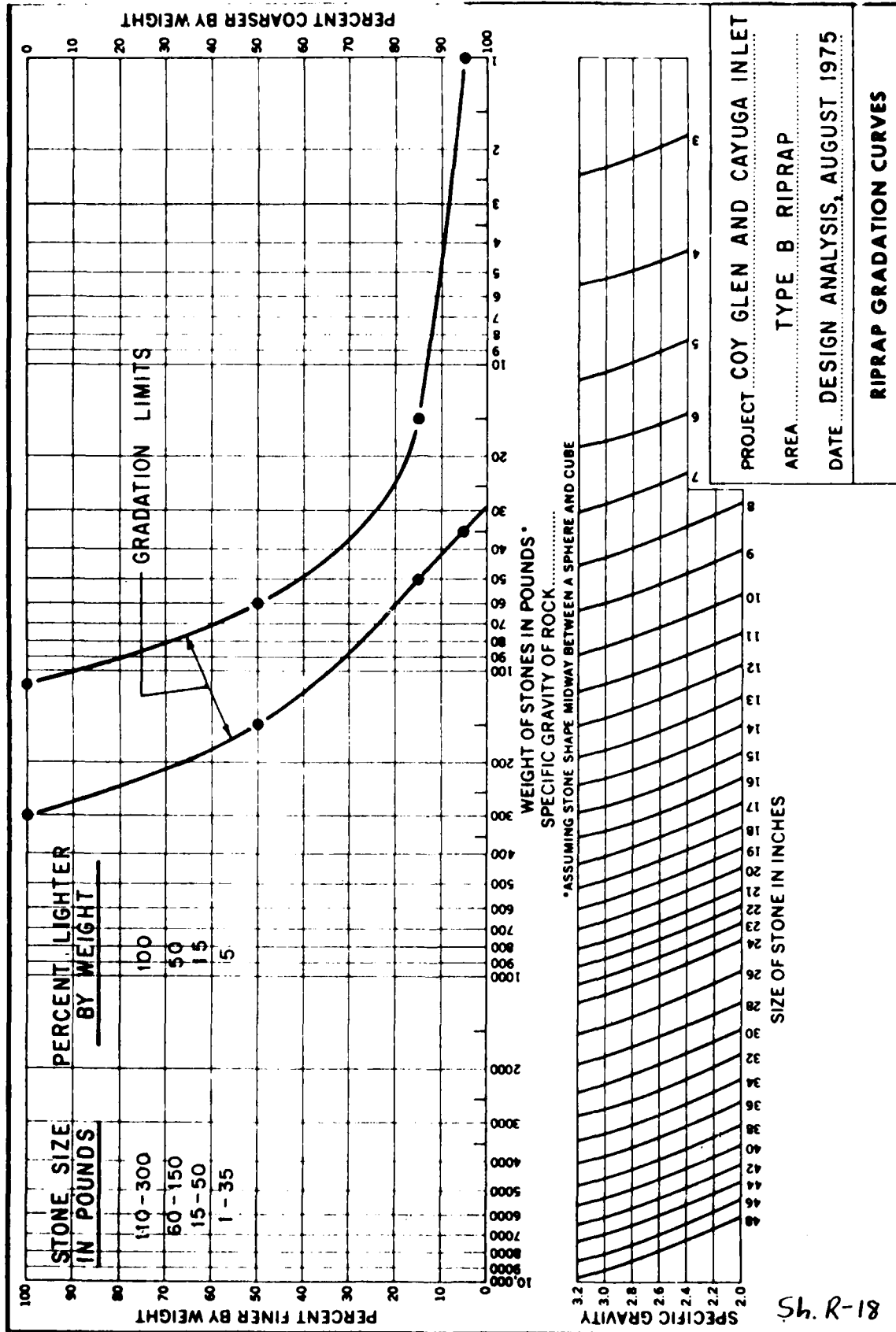
Riprap Gradation			
% by Weight Passing	:	Limits of Stone Weight in Pounds	
	:	Maximum	Minimum
100	:	300	110
50	:	150	60
15	:	50	15

Sh. R-15



Sh. R-16





#### 4. HYDRAULIC DESIGN

4.1 Hydraulic calculations were necessary for the design of the drop structure bottom slab and the design of the baffle blocks. Calculations of the hydraulic impact on the bottom slab are on Sheets H-1 and 2. Calculations of the lateral dynamic force on the baffle blocks are on Sheet H-3.

4.2 The Buffalo District Corps of Engineers developed the hydraulic design for Coy Glen which are on pages 132 through 146 and include the following items:

Hydraulic Design Methodology - page 132

Hydraulics of Spillways - page 137

Drop Structures and Check Dams - page 142

Drop Structure Correspondence - page 145

CALCULATIONS FOR Nappe Impingement on Base  
Dynamic Load Development

From Appendix D, Enclosure U-1 to Contract Documents:

- $Q$  = Discharge in cfs: 500
- $D_u$  = Upstream depth in channel: 4.9' (minimum)
- $I$  = Channel Base Width Upstream: 15'
- $K$  = Side Slope: 2:1
- $A$  = Cross sectional Area
- $D$  = Fall from invert to invert: 10.5' Maximum

The approach velocity of flow over the crest will be increased somewhat due to vertical walls and anticipated flow contraction. If contraction neglected  $V = \frac{500}{15(4.9)} = 6.80$

$h_e$  = Approach velocity head:  $\frac{V^2}{2g}$

For path of impingement, Figure 288-D-2A37 page 411, "Design of Small Dams", 2<sup>nd</sup> Edition, 1973 has been used. Notation follows accordingly, until otherwise indicated.

- $H_e$  = Depth upstream: 4.9'
- $h_d$  = Surface upstream to surface downstream: 9.5 max
- $X$  = Crest to base: 10.5' max
- $g$  = Acceleration due to gravity: 32.2 ft/sec<sup>2</sup>
- $q$  = Discharge / unit width: 500/15 = 33.33 cfs/ft
- $V$  = 33.33/4.9 = 6.80 fps ignoring end contractions

# HNTB

CALCULATIONS FOR

MADE BY SIM DATE 4/2/75 JOB NO. 4204-94-01  
 CHECKED BY SJK DATE 4/3/75 SEC. NO.         
 SHEET NO. H-2

Drop Number  $\bar{D} = \frac{Q^2}{9Y^3} = \frac{33.33^2}{32.2(10.5)^3} = 0.03$   
 $h_0/H_c = 9.5/49 = 1.94$

Froude Number  $F_1 = \frac{V_1}{14.9} = \frac{6.80}{14.9(32.2)} = 0.54$

From 288-D-2437

$L_p/Y = 1.2$

$L_p = 1.2(10.5) = 12.6'$

cross assumptions -

1. No hope contraction in width of plunge  
 equals depth = 9.9'

2. Full impact on base of structure

3. Full velocity buildup due to gravity

4. Time of Travel =  $12.6/6.8 = 1.85$  seconds

5. Vertical velocity =  $\sqrt{2gt^2} = 16.1(1.85)^2 = 55.1 \text{ fps}$

According to Design of Flood Walls a conservative  
 assumption of dynamic force =  $2WA(S.E.)$  where  
 $S.E.$  = specific energy,  $w$  = unit weight of water,  
 $A$  = impingement area

Force =  $\frac{2(62.4)(55.1)^2}{64.4} = 5853 \text{ k/ft}^2 \text{ say } 6000 \text{ psf}$

Deletion of

Subject Cox GlenPage      of      pages.Sheet H-2AComputation of     Computed by AJAChecked by RTGDate 1/12/76

Assume all water falls from the average height of the upstream water depth above the crest down to the top of the base slab.

$$\text{Average height above crest} = \frac{4.9'}{2} = 2.45'$$

$$\text{Distance from crest to top of slab} = 10.50'$$

$$\text{Total average fall} = 12.95' \text{ use } 13'$$

$s = \frac{1}{2} a t^2$  where:  $s$  = fall distance,  $a$  = acceleration due to gravity,  $t$  = time

$$t = \sqrt{\frac{2s}{a}} = \sqrt{\frac{2 \times 13}{32.2}} = .90 \text{ seconds}$$

$v = at$  where  $v$  = velocity

$$= 32.2 \text{ 'sec}^2 \times .90 \text{ sec} = 29.0 \text{ 'sec}$$

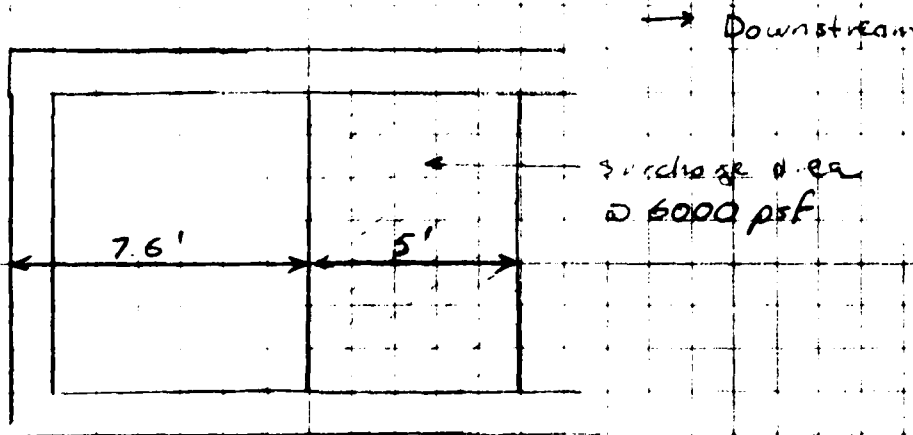
According to "Design of small Dams" a conservative approximation of Dynamic force =  $2 w A (S.E.)$  where  $S.E.$  = specific energy  $\left(\frac{v^2}{2g}\right)$   $w$  = unit weight of water

$A$  = Impingement Area

$$\therefore \frac{\text{Force}}{\text{Area}} = \frac{2(62.4)(29)^2}{64.4} = 1,630 \text{ psf}$$



Location of dynamic surcharge



PLAN - NOT TO SCALE

## LOAD ON IMPACT BLOCKS

"Design of Small Dams" suggests an impinging force on the upstream face of baffle blocks as follows:

$$Force = 2wA(d_1 + h_{v1})$$

where:  $w$  = unit weight of water

$A$  = upstream face area

$(d_1 + h_{v1})$  = Specific Energy entering the basin

Considering 10 block face  $H \times E$  (D-1 spec)  $2.6 \times 1.3$   
 $= 3.38 \text{ ft}^2$

$$1) d_1 = 4.9 \text{ ft}$$

$$3) v_1 = 6.80 \text{ ft/s} \quad h_{v1} = 0.22 \text{ ft} \quad SE = 5.62 \text{ ft}$$

$$\text{Horiz } F \text{ on each block} = 2(62.4) 3.38 (5.62) = 2370$$

Sur 3000 #

Page 1 of 3 pages  
 Subject CCY GLENN, ITALY, N.Y. - 50 YR DESIGN Q = 500 CFS  
 Computation of OBJECTIVES & CONCLUSIONS  
 Computed by B.S.P. Checked by \_\_\_\_\_ Date 17 June 72

SCHEME - 1: 1A - 2: 2A  
 THE PURPOSE OF THIS HYDRAULIC DESIGN IS TO  
 PROVIDE STABILIZATION TO THE COYGLENN DRAINAGE BASIN.  
 CRITICAL SLOPES HAVE CAUSED EXCESSIVE VELOCITIES  
 AT LOW FLOWS AND CONSEQUENTLY THE CHANNEL HAS  
 BECOME SEVERELY ERODED. TWO SCHEMES <sup>FOR</sup> SOLVING  
 THE PROBLEM WILL BE EXAMINED.

1] USING TWO DROP STRUCTURES TO DISSIPATE ENERGY

2] ONE DROP STRUCTURE TO DISSIPATE ENERGY

RESULTS ARE: SCHEME 1

1] TWO IDENTICAL DROP STRUCTURES @ STA 2+00  
 AND 3+28 WOULD BE NEEDED. DIMENSIONS  
 ARE:

LBASIN = 24 FT  
 LTOTUS. FACE OF BLOCKS = 18.2 FT SEE SHEET 1  
 BLOCK HT. = 2.6 FT FOR BACKWATER  
 WIDTH & SPACING = 13 FT  
 END SILL HT = 2 FT  
 Y = 10.5 FT [SEE PG. 2-4]

SCHEME 2

2] ONE DROP STRUCTURE AT STA 1+22.5

LBASIN = 27.5 FT  
 LTOTUS. FACE OF BLOCKS = 21.8 FT SEE SHEET 1  
 BLOCK HT = 2.6 FT FOR BACKWATER  
 WIDTH & SPACING = 13 FT  
 END SILL HT = 2 FT  
 Y = 19.1 FT

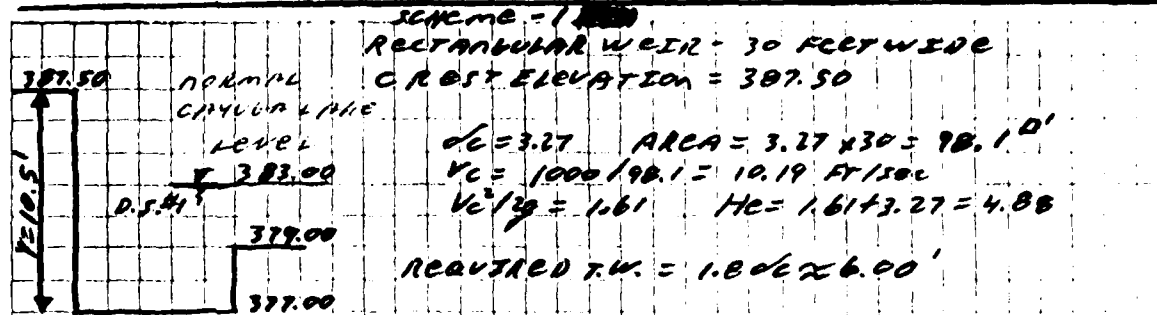
SCHEME - 1-A

1] TWO D.S. AS IN SCHEME 1 ONLY THE CHANNEL BOTTOM IS  
 ONLY 15' WIDE - 50 YEAR DESIGN Q = 500 CFS

SCHEME - 2-A

1] ONE D.S. AS IN SCHEME 2 ONLY THE CHANNEL BOTTOM IS  
 ONLY 15' WIDE - 50 YEAR DESIGN Q = 500 CFS

Subject COTY CULVERT, ITHACA N.Y. - 100 YR DESIGN Q = 1000 CFS  
 Computation of BASED DIMENSIONS - CONCRETE D.S. AT STA 0+00  
 Computed by B.S.A. Checked by \_\_\_\_\_ Date 17 JUNE 72



a] ENERGY LINE ELEVATION AT CREST OF DROPP = CREST ELEV +  $h_c + \frac{V_c^2}{2g}$   
 $= 387.50 + 3.27 + 1.61 = 392.38$

b] T.W. ELEVATION = 383.00  $h_c = 392.38 - 383.00 = 9.38$

c]  $\frac{h_c}{H_E} = \frac{9.38}{4.88} = 1.92$  d] DROPP =  $\bar{D} = \frac{Q^2}{9V^3}$

e]  $Q = 1000 / 30$   $\bar{D} = \frac{[1000/30]^2}{[32.2][10.5]^3} = 0.03$

f]  $Y = 387.50 - 377.00 = 10.5$  FT

FROM CURVE, PG 309 OF "DESIGN OF SMALL DAMS"

FOR  $\bar{D} = 0.03$  AND  $\frac{h_c}{H_E} = 1.92$

$L_p / Y = 1.3 \rightarrow L_p = 10.5 \times 1.3 = 13.65$

$L_{BASED} = L_p + 2.55 d_c = 13.65 + 8.34 = 22.0$  FT

UPR. U.S. FACE =  $L_p + 1.80 d_c = 13.65 + 2.62 = 16.27$  FT  
 OF BLOCKS

BLOCK HT =  $0.80 d_c = 2.6$  FT

WIDTH / SPACING =  $0.4 d_c = 1.3$  FT

GO TO PAGE 3

NOTE - THESE CALCULATIONS ARE ALSO FOR SCHEME 1-A  
 $Q = 500$  CFS

Subject COY LKAR, ETHAL P. D. Y. - 100 YR DESIGN Q = 1000 CFS  
 Computation of BASED DIMENSION - AFTER EM 1110-345-284  
 Computed by B.S.P. Checked by \_\_\_\_\_ Date 17 June 72

SCHEME - 1: 1A

$h = 8.5$   $dc = 3.27$

$h/dc = 8.5 / 3.27 = 2.60 \Rightarrow$  FROM FIG 24-B 64  $\frac{h'}{dc} = 0.50$

FOR  $\frac{h}{dc} = 2.60 \Rightarrow CL = 3.8 = \frac{L}{\sqrt{HGE}}$  SO  $h' = 0.5 \times 3.27 = 1.63$

$L = 3.8 \times \sqrt{0.5 \times 3.27} = 3.8 \times \sqrt{1.635} = 3.8 \times 1.279 = 4.86$

REDUCE 20.02 BY 10% =  $20.02 / 1.1 = 18.2$  FT

FROM DESIGN OF S.D. - LENGTH TO BAFFLES = 16.2 FT  
 " EM 1110-345-284 - " " " = 18.2 FT

SO BASIN SHOULD BE 2.0 FT LONGER THAN THAT DETERMINED FROM DESIGN OF S.D.

$L_B = 22 + 2 = 24$  FT

NOTE - USE SAME SIZE UNAP STRUCTURE AT STA 1+24.  
 THAT IS;

$L_{BASIN} = 24$  FT

$L_{TO U.S. FACE OF BUCKLE} = 18.2$  FT

BLOCK HT = 2.6 FT

WIDTH & SPACING = 1.3 FT

END SILL HT = 2 FT

FLOW LINE COMPUTATIONS UNDER IMPROVED CONDITIONS										STREAM		SHEET NO. 1 of 2							
DISCHARGE										COY Glen, Itasca, N.Y.		DATE 7/10/75							
52000 = 0.005										Q = 500 C.F. Per Sec.									
CHANNEL BOTTOM ELEVATION	DEPTH OF WATER	MILE ON STATION	LENGTH OF REACH $\frac{L}{100}$	TOTAL ELEVATION	AREA	BOTTOM WIDTH	HYD RADIUS	COEF. OF ROUGH	VELOCITY	$\frac{Q}{A}$	SLOPE	MEAN SLOPE	VELOCITY	DISCHARGE	PRIVISION HEAD	VELOCITY HEAD	TOTAL HEAD	WATER SURFACE ELEVATION	ENERGY GRADIENT
378.10	2.85	0.400	—	380.95	57	15.12	2.12	0.045	0.55	32.45	0.01442	0.01442	8.47	500	—	1.16	—	380.95	382.01
378.05	3.70	0.410	10	381.75	83	15.12	2.63	0.045	0.63	52.29	0.00747	0.00747	5.02	500	0.16	0.56	381.73	382.29	
378.10	3.80	0.420	10	381.90	86	15.12	2.68	0.045	0.64	55.04	0.007252	0.007252	5.81	500	0.09	0.53	381.85	382.38	
378.20	3.90	0.440	20	382.10	89	15.12	2.74	0.045	0.65	57.85	0.007470	0.007470	5.62	500	0.16	0.49	382.05	382.54	
378.00	4.20	0.400	100	382.20	98	15.12	2.91	0.045	0.67	65.66	0.005735	0.005735	5.10	500	1.06	0.40	382.22	383.62	
378.00	5.00	0.400	—	383.00	125	15.12	3.35	0.045	0.74	92.5	0.00292	0.00292	4.00	500	—	0.25	—	383.00	383.25
378.50	4.80	0.400	100	383.30	118	15.12	3.24	0.045	0.72	84.96	0.007470	0.007470	4.24	500	0.72	0.28	383.30	383.58	
379.00	4.70	0.400	100	383.70	115	15.12	3.18	0.045	0.72	82.8	0.007470	0.007470	4.35	500	0.76	0.29	383.75	383.94	

COMPUTED BY  
B.S.R.

CHECKED BY

Q = 14'  $\frac{1}{2}$  = 8mL

S =  $\left( \frac{Q}{A} \right)^{0.148}$

where  $\frac{Q}{A}$  = 0.01

R = 1/7

NOTES

Schematic - fill above #1 Drop Structure to elev 397.00										SHEET NO. 2 of 2										
STREAM Coyle Creek										DATE July 1, 1975										
DISCHARGE Q = 300 cfs P. Per. Sec.																				
FLOW LINE COMPUTATIONS UNDER IMPROVED CONDITIONS																				
Slope STA 2428 to 3428 = 0.01																				
Slope STA 3456 to 6400 = 0.00																				
Slope STA 6400 to 7400 = 0.00																				
Notes: Natural																				
CHANNEL BOTTOM ELEVATION	DEPTH OF WATER	MILE ON STATION	LENGTH OF REACH	TRIAL ELEVATION	AREA	BOTTOM WIDTH	HYD. RADIUS	COEF. OF ROUGH.	$\frac{S}{V^2} = .01$	VELOCITY	DISCHARGE	SLOPE	MEAN SLOPE	VELOCITY	DISCHARGE	FRICTION HEAD	VELOCITY HEAD	TOTAL HEAD	WATER SURFACE ELEVATION	ENERGY GRADIENT
397.5	3.27	2428	5	390.77	49.05	15.18	2.28	.035	.74	36.30	0.1897	.0105		10.19	500	.05	1.61	250.77	392.38	
397.55	4.30	2433	10	392.35	118	15.12	3.24	.035	.93	110	.00209	.00205		4.24	500	.03	.28	1.65	392.42	392.70
397.65	4.70	2443	35	392.45	118	15.12	3.24	.035	.93	110	.00205	.00205		4.24	500	.03	.28	1.65	392.44	392.72
397.50	4.10	2428		392.42	95.1	15.12	2.85	.035	.85	90.9	.00223	.00223		5.26	500		.43	1.10	392.54	392.97
START at Critical Depth of the Crest of the Drop Structure																				
397.00	2.27	2456	10	390.21	49.05	15.18	2.28	.035	.74	36.30	0.1897	.00962		10.19	500	.10	1.61	250.27	400.27	401.89
397.00	3.30	2466	34	402.30	268	15.12	4.21	.035	1.11	297	.000253	.000253		1.87	500	.01	.03	1.97	402.24	
397.00	5.30	4500	100	402.30	268	15.12	4.21	.035	1.11	297	.000253	.000253		1.87	500	.01	.03	1.97	402.25	
397.00	5.30	4500	100	402.30	268	15.12	4.21	.035	1.11	297	.000253	.000253		1.87	500	.01	.03	1.97	402.28	
397.00	5.30	4500	100	402.30	268	15.12	4.21	.035	1.11	297	.000253	.000253		1.87	500	.01	.03	1.97	402.31	
400.50	2.10	5500	100	402.60	92.9	15.12	1.89	.045	.51	47.3	0.11610	.005815		5.39	500	.58	.45	4.94	402.65	
402.00	1.55	8400	100	404.55	16.80	15.12	1.43	.045	.42	20.6	.0317625	.021460		7.48	500	2.15	.87	4.61	404.55	
405.00	1.50	4500	100	407.50	17.10	15.12	1.43	.045	.42	20.6	.0317625	.020735		5.12	500	2.07	.87	4.25	407.55	
NOTES: B = MP $V = \frac{1.486}{S^{.0167}} R^{.74}$ where R = ft $Q = AV$ $b_f = 8ml$										COMPUTED BY: 41										
										CHECKED BY: 51										

structural design. The structure must be made sufficiently stable to resist sliding against the impact load on the baffle wall. The entire structure must resist the severe vibrations inherent in this type of device, and the individual structural members must be sufficiently strong to withstand the large dynamic loads.

Riprapping should be provided along the bottom and sides adjacent to the structure to avoid the tendency for scour of the outlet channel downstream from the end sill when a shallow tailwater exists. Downstream wingwalls placed at 45° may also be effective in reducing scouring tendencies and flow concentrations downstream.



Figure 218. An impact type stilling basin in operation.

**203. Plunge Pools.**—When a free-falling overflow nappe drops vertically into a pool in a riverbed, a plunge pool will be scoured to a depth which is related to the height of the fall, the depth of tailwater, and the concentration of the flow [13]. Depths of scour are influenced initially by the erodibility of the stream material or the bedrock and by the size or the gradation of sizes of any armoring material in the pool. However, the armoring or protective surfaces of the pool will be progressively reduced by the abrading action of the churning material to a size which will be scoured out and the ultimate scour depth will, for all practical considerations, stabilize at a limiting depth irrespective of the material size. An empirical approximation based on experimental data has been developed by Veronese [14] for limiting scour depths, as follows:

$$d_s = 1.32 H_T^{0.225} q^{0.54} \quad (26)$$

where,

$d_s$  = the maximum depth of scour below tailwater level in feet,

$H_T$  = the head from the reservoir to tailwater levels in feet, and

$q$  = the discharge in second-feet per foot of width.

## F. HYDRAULICS OF SPILLWAYS

### 204. Free Overfall (Straight Drop) Spillways.

(a) *General.* The hydraulic problems of the free overfall spillway are concerned with the characteristics of the control and with the dissipation of flow in the downstream basin. Flow over the control ordinarily is free discharging; air is admitted to the underside of the nappe to avoid the jet being depressed by reduced underneath pressure. Dissipation of the flow in the downstream basin may be obtained by the hydraulic jump, by impact and turbulence induced in a basin with impact blocks, or by a slotted grating dissipator installed immediately downstream from the control.

The control may be either sharp crested to provide a fully contracted vertical jet, broad crested to effect a fully suppressed jet, or shaped to increase the crest efficiency. Coefficients of discharge will approximate those indicated in section 190. The sides of the control usually are arranged to allow for full side contraction, in order to provide side space for the access of air to the underside of the nappe. This contraction is effected by providing square abutment headwalls or by installing square-cornered vertical offsets along the piers or walls opposite the crest. The effective length of the crest is then determined according to

equation (4) where  $K_p$  and  $K_a$  will approximate 0.20.

The dimensions of the stilling basin for the free overfall spillway can be related to two independent variables; namely, the drop distance  $Y$  and the unit discharge  $q$ . These variables, which are dimensional terms, can be expressed in a dimensionless ratio by expressing  $q$  in lineal form by means of the equation for critical depth,

$d_c = \sqrt[3]{\frac{q^2}{g}}$ , as follows:

$$\frac{d_c}{Y} = \sqrt[3]{\frac{q^2}{gY^3}}$$

From this expression it can be seen that  $\frac{q^2}{gY^3}$  is a dimensionless ratio which can be used as an independent variable to which the individual dimensions may be related. This ratio is called the "drop number" and is designated  $\bar{D}$ . It can be shown that  $\bar{D}$  is the product of  $F_1^2$  and  $\left(\frac{d_1}{Y}\right)^3$ ,

where  $F_1$  is the Froude number  $\frac{v_1}{\sqrt{d_1 g}}$  at the point where the nappe meets the basin floor.

(b) *Hydraulic Jump Basin*.—The jump characteristics of the straight drop basin are basically the same as those for other jump basins, except that the position of the start of the jump cannot be determined as readily as it can for other basins. On figure 219 the point of the start of the jump (point X) will vary with the vertical drop distance and is influenced by the under nappe pool depth,  $d_f$ . The basin design downstream from point X will be patterned after those discussed in section 199, once distance  $L_a$  is determined. Values of the depth  $d_1$ , and of the Froude number,  $F_1$ , at the start of the jump in relation to the drop number,  $\bar{D}$ , are shown on figure 219. These relations may be used for determining the basin dimensions.

Where tailwater depths are greater than the conjugate depth  $d_2$ , the jump will move back on the free falling nappe raising the depth  $d_f$  of the under nappe pool. With greater depths of the under nappe pool, the nappe will not plunge immediately to the floor of the basin but will be deflected upward along the top of the under pool so that it will meet the floor to the right of point X. The distance to the start of the jump,  $L_a$ , will become progressively longer as the tailwater

depth is increased. Average values of  $L_a$  in relation to  $\frac{h_a}{H_c}$ , as determined from tests, are plotted on figure 219. For a basin with excessive depths the type II basin discussed in section 199 is most adaptable. The impact block type basin, discussed below, also can be adopted for low drop spillways with excessive tailwater depths.

(c) *Impact Block Type Basin*. An impact block basin has been developed [1] for low heads which gives reasonably good dissipation of energy for a wide range of tailwater depths. The dissipation of the high energy is principally by turbulence induced by the impingement of the incoming flow upon the impact blocks. The required tailwater depths, therefore, become more or less independent of the drop height. The linear proportions are as follows:

Minimum basin length,  $L_b = L_p + 2.55 d_c$

Minimum length to upstream face of baffle block  $= L_p + 0.8 d_c$

Minimum tailwater depth,  $d_{tw} = 2.15 d_c$

Optimum baffle block height  $= 0.8 d_c$

Width and spacing of baffle block  $= 0.4 d_c \pm$

Optimum height of end sill  $= 0.4 d_c$

(d) *Slotted Grating Dissipator*.—An effective dissipator for small drops is illustrated on figure 220. This device has been tested for values of the Froude number,  $F_1$ , as determined at basin apron level, in the range of 2.5 to 4.5. For this arrangement the overfalling sheet is separated into a number of long, thin segments which fall nearly vertically into the basin below, where dissipation of energy takes place by turbulence. To be effective the length of the grating,  $L_g$ , must be such that the entire incoming flow will fall through the slots before reaching the downstream end. The length is therefore a function of the total discharge, the velocity of the incoming flow, and the area of the grating slots. Experimental tests indicate that the following relation gives an effective design:

$$L_g = \frac{Q}{0.245 w N \sqrt{2gH_c}} \quad (27)$$

where:

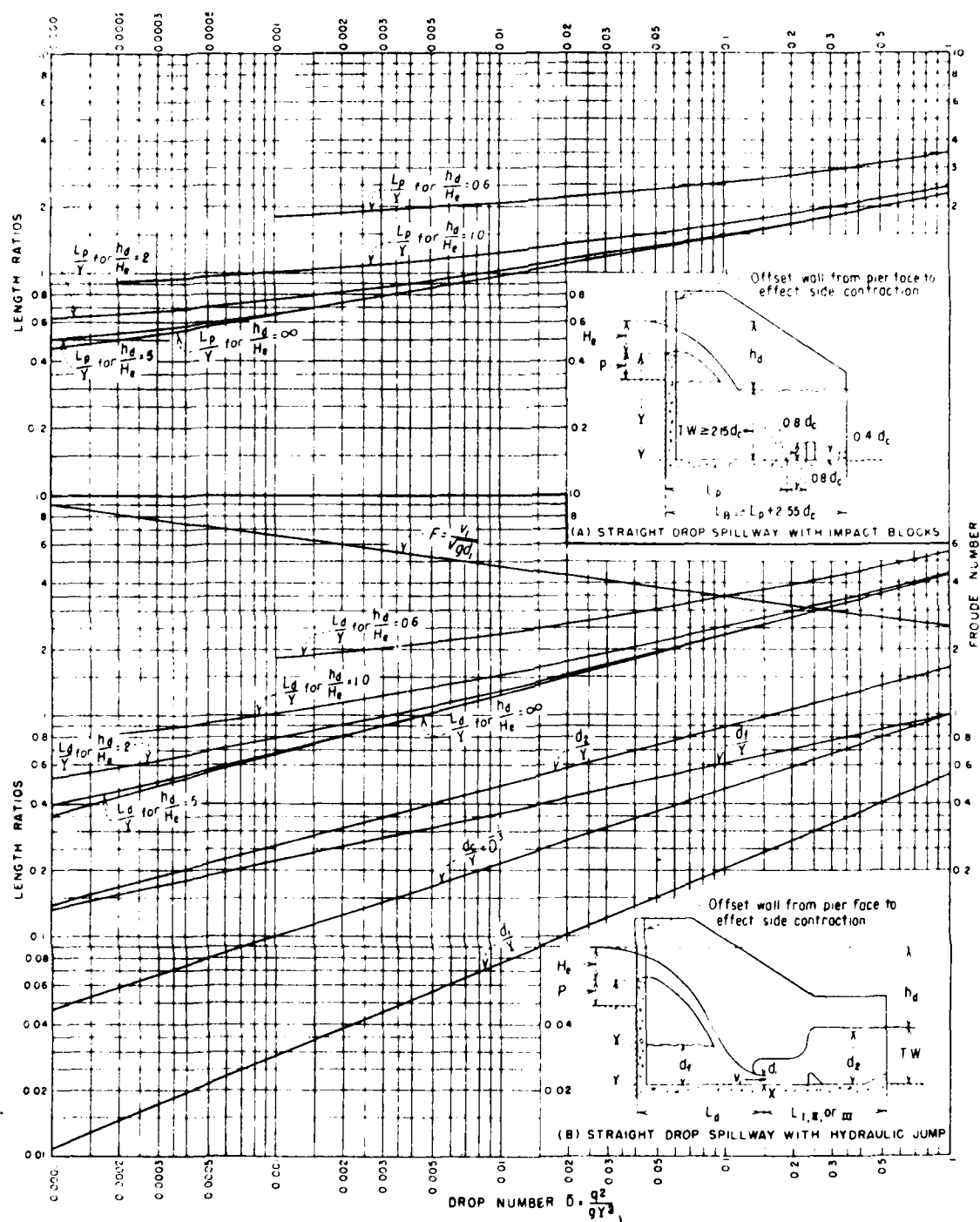
$L_g$  = the length of the grating in feet,

$w$  = the width of the slot in feet,

$N$  = the number of slots, and

$H_c$  = the depth of flow upstream from the drop.





**Figure 219. Hydraulic characteristics of straight drop spillways with hydraulic jump or with impact blocks.**

The length of the basin,  $L_B$ , should be approximately  $1.2 L_c$ . An end sill similar to that for basin type I, discussed in section 199, can be provided to improve the hydraulic action.

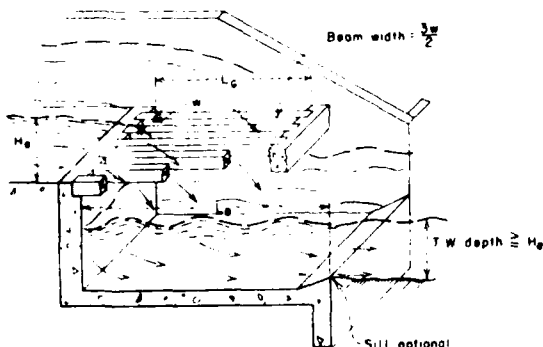


Figure 220. Slotted grating dissipator.

(c) *Example of Design of a Free Overfall Spillway.* The procedure for designing a free overfall spillway is best shown by means of an example. Consider that such a spillway is to be designed to discharge 500 second-feet. The drop from the spillway crest to the tailwater level for a flow of 500 second-feet is 12 feet. (Tailwater elevation is 108.0.) The approach channel is 20 feet long and the approach floor is level with the spillway crest which is at elevation 120.0. Each type of energy dissipator is to be investigated.

The procedure for design of the *hydraulic jump basin* is as follows: First, assume the effective length of the spillway crest to be 15 feet. Assume an approximate value of  $C=3.0$ . The unit discharge,  $q$ , is equal to  $\frac{500}{15}=33.3$  second-feet and  $H_c$  is equal to  $\left(\frac{q}{C}\right)^{2/3}=\left(\frac{33.3}{3.0}\right)^{2/3}=5.0$  feet. The reservoir water surface elevation, therefore, is  $120.0+5.0=125.0$ . Thus the drop from reservoir level to tailwater level will be approximately 17 feet.

Assume that an offset of 0.5 foot is provided along each side of the weir to effect side contractions for aerating the underside of the sheet, and that the offset is square-cornered. Then the net crest length, which will also be the stilling basin width, is:

$$L' = L + 2K_d H_c + 2(0.5) = 15 + 2(0.2)(5) + 1.0 = 18.0 \text{ feet.}$$

Figure 208 is used to determine the approximate apron level of the jump basin, assuming the effective width of the basin to be 15 feet and (for the first trial) that there will be no loss of energy between the reservoir and the point where the jet strikes the basin floor. From scale A, the conjugate depth  $d_2$  for  $q=33.3$  second-feet and  $H_T=17$  feet is 8.8 feet, which places the apron floor at elevation  $99.2$ .  $Y$  is equal to elevation  $120.0 - \text{elevation } 99.2 = 20.8$  feet, and the drop number  $\bar{D}$  is equal to  $\frac{q^2}{gY^3} = \frac{33.3^2}{32.2 \times 20.8^3} = 0.0038$ . For  $\bar{D} =$

$0.0038$ , from figure 219  $\frac{d_2}{Y} = 0.375$  and  $d_2 = 7.8$  feet. The apron level then must be adjusted to an elevation which is  $d_2$  below the tailwater elevation 108.0, or elevation 100.2.

For the second trial, the adjusted value of  $Y$  is 19.8 and  $\bar{D}$  is equal to  $\frac{33.3^2}{32.2 \times 19.8^3} = 0.0044$ . For  $\bar{D} = 0.0044$  and  $\frac{h_d}{H_c} = \frac{17}{5} = 3.4$ , from figure 219,  $\frac{L_d}{Y} = 1.02$  and  $L_d = 20.2$  feet. Also  $d_1 = 1.1$  feet and  $F_1 = 5.3$ .

With the values of  $F_1 = 5.3$ ,  $d_1 = 1.1$  and  $d_2 = 7.8$ , the arrangement of the type II basin shown on figure 206 can be used. From figure 206,  $\frac{L_{II}}{d_2} = 2.37$  and  $L_{II} = 18.5$  feet. The length of the basin measured from the vertical crest is equal to  $L_d + L_{II} = 20.2 + 18.5 = 38.7$  feet. The distance of the baffle blocks from the vertical crest for this basin will be 20.2 feet plus  $0.8 d_2$  or 20.2 plus  $0.8 (7.8) = 26.4$  feet, approximately.

The baffle blocks will be approximately 1.5  $d_1$  or 1.6 feet high and will be about 14 inches wide and spaced at about 28-inch centers.

For the *impact block basin*, the procedure is as follows: The critical depth,  $d_c$ , is equal to  $\sqrt[3]{\frac{q^2}{g}} = \sqrt[3]{\frac{33.3^2}{32.2}} = 3.3$  feet. Then from figure 219, for  $\bar{D} = 0.0044$  and  $\frac{h_d}{H_c} = 3.4$ ,  $\frac{L_p}{Y} = 0.85$  and  $L_p = 17.0$  feet. The minimum length of the basin,  $L_B$ , is equal to  $L_p + 2.55 d_c = 17.0 + 2.55 (3.3) = 25.4$  feet, say 26 feet. The minimum tailwater depth of  $2.15 d_c$  will be 7.1 feet which places the basin

floor at elevation 100.9. The distance from the vertical crest to the baffle blocks will be  $L + 0.8 d_c = 17.0 + 0.8 (3.3) = 19.6$  feet, say 20 feet. The baffle blocks will be about  $0.8 d_c$  or 3.0 feet high and about 18 inches wide, spaced at about 3-foot centers. The end sill will be  $0.4 d_c$  or about 1.5 feet high.

It can be seen from the above result that if the impact block basin is used, the basin can be made almost 13 feet shorter than that required for a hydraulic jump basin, and also that the impact block basin will be 0.7 foot shallower. The baffle blocks for the hydraulic jump basin will be smaller and spaced closer together than those for the impact block basin.

This example shows that the impact block basin is considerably smaller than the hydraulic jump basin. However, the impact block basin should be limited to use where the drop distance does not exceed 20 feet. Furthermore, as previously explained, the foundation for an impact block basin must be of better quality because of the concentrated forces involved. The hydraulic jump basin, therefore, has a much wider application of use.

The *slotted grating dissipator* is not suitable in this case because the Froude number of 5.3 is in excess of the 4.5 value, which is the tested limit for a practical slotted grating design.

**205. Drop Inlet (Shaft or Morning Glory) Spillways.**—(a) *General Characteristics.*—Typical flow conditions and discharge characteristics of a drop inlet spillway are represented on figure 221. As illustrated on the discharge curve, crest control (condition 1) will prevail for heads between the ordinates of  $a$  and  $g$ ; orifice or tube control (condition 2) will govern for heads between the ordinates of  $g$  and  $h$ ; and the spillway conduit will flow full for heads above the ordinate of  $h$  (represented as condition 3).

Flow characteristics of a drop inlet spillway will vary according to the proportional sizes of the different elements. Changing the diameter of the crest will change the curve  $ab$  on figure 221 so that the ordinate of  $g$  on curve  $cd$  will be either higher or lower. For a larger diameter crest, greater outflows can be discharged over the weir at low heads and the transition will fill up and tube control will occur with a lesser head on the crest. Similarly, by altering the size of the

throat of the tube, the position of curve  $cd$  will change, indicating the heads above which tube control will prevail. If the transition is made of such size that curve  $cd$  is moved to coincide with or lie to the right of point  $j$ , the control will shift directly from the crest to the downstream end of the conduit. The details of the hydraulic flow characteristics are discussed in following subsections.

(b) *Crest Discharge.*—For small heads, flow over the drop inlet spillway is governed by the characteristics of crest discharge. The vertical transition beyond the crest will flow partly full and the flow will cling to the sides of the shaft. As the discharge over the crest increases, the overflowing annular nappe will become thicker, and eventually the nappe flow will converge into a solid vertical jet. The point where the annular nappe joins the solid jet is called the crotch. After the solid jet forms, a "boil" will occupy the region above the crotch; both the crotch and the top of the boil become progressively higher with larger discharges. For high heads the crotch and boil may almost flood out, showing only a slight depression and eddy at the surface.

Until such time as the nappe converges to form a solid jet, free-discharging weir flow prevails. After the crotch and boil form, submergence begins to affect the weir flow and ultimately the crest will drown out. Flow is then governed either by the nature of the contracted jet which is formed by the overflow entrance, or by the shape and size of the vertical transition if it does not conform to the jet shape. Vortex action must be minimized to maintain converging flow into the drop inlet. Guide piers are often employed along the crest for this purpose [5, 6, 22].

If the crest profile and transition conform to the shape of the lower nappe of a jet flowing over a sharp-crested circular weir, the discharge characteristics for flow over the crest and through the transition can be expressed as:

$$Q = C L H^{3/2} \quad (3)$$

where  $H$  is the head measured either to the apex of the under nappe of the overflow, to the spring point of the circular sharp-crested weir, or to some other established point on the overflow. Similarly,

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energy dissipators apply also to storm-drain outfalls. Generally, however, the range of exit velocities is likely to be more limited for storm drains, and elaborate structures for energy dissipation are rarely required. If the storm drain discharges into a large stream channel or a lake or ocean where strong hydraulic forces are present, artificial dissipation of effluent energy is rarely required, but particular care must be taken to insure that the outlet structure (1) does not adversely affect the streambank or shore stability, and (2) is not caused to fail as a result of the exterior forces.

c. Channel Outlets. Improved channels, especially the paved ones, commonly carry water at velocities higher than those prevailing in the natural channels into which they discharge. Usually a judicious use of riprap will suffice for dissipation of excess energy. The terminus of a paved channel will require a cutoff wall to preclude undermining. In extreme cases a structure such as a flared transition, stilling basin, or impact device may be required.

2-14. DROP STRUCTURES AND CHECK DAMS. a. Description and Purpose. Drop structures and check dams are designed to check channel erosion by controlling the effective gradient, and to provide for abrupt changes in channel gradient by means of a vertical drop. They also provide a satisfactory means for discharging accumulated surface runoff over fills with heights not exceeding about 5 feet and over embankments higher than 5 feet provided the end sill of the drop structure extends beyond the toe of the embankment. The check dam is a modification of the drop structure used for erosion control in small channels where a less elaborate structure is permissible. The hydraulic design of these structures may be divided into two general phases, design of the notch or weir and design of the overpour basin. It must be emphasized that for a drop structure or check dam to be successful, not only must the structure be designed soundly, but also the structure or series of structures must be so placed as to cause the ditches or channels to become stable. The question of what is a stable grade for the channel must be answered before the height and spacing of the various drop structures can be determined. Also the structure must be designed to preclude flanking.

b. Design Rules. Pertinent features of a typical drop structure are shown in figure 24. (Features of an alternate drop structure are given in paragraph DROP STRUCTURES AND CHECK DAMS of EM 1110-345-283.)

(1) Notation used in the design of drop structures is as follows:

$C$  = weir coefficient = 3.0

$C_L$  = coefficient of basin length =  $\frac{L}{\sqrt{hd_c}}$

$d_c$  = critical depth over crest, feet

$H$  = head on weir =  $3/2 d_c$ , feet

$h$  = height of drop, feet

$h'$  = height of end sill, feet

$L$  = length of basin, feet

$Q$  = discharge, cubic feet per second

$W$  = length of weir or width of crest, feet

(2) Weir. Discharge over the weir should be computed from the equation  $Q = CWH^{3/2}$ , using a  $C$  value of 3.0. The length of the weir should be such as to obtain maximum use of the available channel cross section upstream from the structure. A trial-and-error procedure should be used to balance the weir height and width with the channel cross section.

(3) Stilling basin length and end sill height should be determined from the design curves in figure 24.

(4) Riprap probably will be required on the side slopes and below the end sill immediately downstream from the structure.

2-15. MISCELLANEOUS STRUCTURES. a. Chutes. The chute provides a satisfactory method of discharging accumulated surface runoff over fills and embankments. A typical design is presented in figure 25, and design charts for chutes constructed of concrete for various gradients and discharges are shown in figure 26. On the basis of laboratory

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Coy Glen - Drop Structure

$$Q = 1000 \text{ cfs}$$

$$W = 30 \text{ ft}$$

$$q = 33.3 \text{ cfs}$$

$$d_c = 3.25$$

$$h = 393.5 - 386.0 = 7.5 \text{ ft}$$

$$h/d_c = 7.5/3.25 = 2.3$$

From fig. 24, EM 1110-345-284, - *TM 5-BW-4*

$$C_L = 4.0$$

$$L = 4.0 \sqrt{hd_c} = 19.7$$

Since design curve results in a basin length 10 percent greater than minimum acceptable, reduce

$$19.7/1.1 = 17.9 \text{ ft}$$

Thus, I would place the row of baffles 18 ft from the drop rather than 15.6 ft as shown on sketch with letter to Weinrub dated 28 April 1972. If a solid sill is used in place of baffles, its height should be  $0.5 d_c = 1.7 \text{ ft}$ . With baffles, 2.6 ft as shown probably is good.

Also, the above indicate that the basin floor should be at about elev. 384. I would raise the basin to this elevation, use a 2-ft-high end sill, and eliminate the reverse slope on the channel bottom immediately downstream of the end sill. This 1 on 3 reverse slope would require large riprap. If a reverse slope were required, it should be no steeper than 1 on 10.

Further, I would put some rounding on the abutments (see plate 2 of the Gering report, TR 2-760). Also I would terminate the side walls at the end sill. The flared wing walls do more harm than good; use only if required as retaining walls.

Summarizing, I would end up with a basin 24.5 ft long rather than 22 ft, and at elev. 384.0 rather than 382.81. I would place the 2.6-ft-high baffles 18 ft from the drop rather than 15.6 ft. I would use a 2-ft-high end sill. I would round the abutment walls and would eliminate the flared wing walls.

T. E. MURPHY  
Chief, Structures Branch  
15 May 1972



COY GLEN AND CAYUGA INLET  
ITHACA, NY

MATERIAL SURVEY  
FOR  
DESIGN ANALYSIS

GENERAL

1. A materials survey to determine construction material sources for energy dissipator facilities and riprap repair was performed. Interested sources were investigated.
2. The survey consisted of a preliminary file search in which the following were considered:
  - a. An analysis of the results of quarry investigations.
  - b. Laboratory testing of samples and an analysis of the test results, and
  - c. The evaluation of available service records.
3. The survey included a sufficient number of sources capable of producing the required materials.

MATERIAL DESIGN CRITERIA

4. Material Types and Gradations

a. General. The stone materials for the proposed construction consists of two sizes of riprap, spalls, and bedding. In all cases, no stone shall exceed an elongation ratio of 3:1.

b. Type A Stone. (Riprap). This stone will be a reasonably well graded material having a maximum size of 700 pounds. The gradation shall be as follows and shall be within the limits shown on Figure 1 at the end of this section.

Stone Size in Pounds	:	Percent Lighter by Weight
250-700	:	100
135-250	:	50
40-100	:	15
1-80	:	5

c. Type B Stone. (Riprap). This stone will be a reasonably well-graded material having a maximum size of 300 pounds. The gradation shall be as follows and shall be within the limits shown on Figure 2 at the end of this section.

Stone Size in Pounds	:	Percent Lighter by Weight
110-300	:	100
60-150	:	50
15-50	:	15
1-35	:	5

d. Type C Stone (Spalls). This material will consist of a reasonably well-graded stone and shall have sizes ranging between 8 inches and fines. The gradation shall be as follows and shall be within the limits shown on Figure 3 at the end of this section.

Sieve Designation U.S. Standard Square Mesh	:	Percent Finer by Weight
8 inches	:	100
6 inches	:	80-100
3 inches	:	40-70
1 inch	:	0-25
1/2 inch	:	0-10

e. Type D Stone (Bedding). This material will consist of a reasonably well-graded stone ranging between 4 inches and fines. The gradation shall be as follows and shall be within the limits shown on Figure 3 at the end of this section.

c. Type B Stone. (Riprap). This stone will be a reasonably well-graded material having a maximum size of 300 pounds. The gradation shall be as follows and shall be within the limits shown on Figure 2 at the end of this section.

Stone Size in Pounds	:	Percent Lighter by Weight
110-300	:	100
60-150	:	50
15-50	:	15
1-35	:	5

d. Type C Stone (Spalls). This material will consist of a reasonably well-graded stone and shall have sizes ranging between 8 inches and fines. The gradation shall be as follows and shall be within the limits shown on Figure 3 at the end of this section.

Sieve Designation U.S. Standard Square Mesh	:	Percent Finer by Weight
8 inches	:	100
6 inches	:	80-100
3 inches	:	40-70
1 inch	:	0-25
1/2 inch	:	0-10

e. Type D Stone (Bedding). This material will consist of a reasonably well-graded stone ranging between 4 inches and fines. The gradation shall be as follows and shall be within the limits shown on Figure 3 at the end of this section.

Sieve Designation U.S. Standard Square Mesh	:	Percent Finer by Weight
4 inches	:	100
2 inches	:	65-100
1 inch	:	50-90
3/4 inch	:	45-83
No. 4	:	25-60
No. 10	:	14-48
No. 40	:	0-30
No. 200	:	0-10

5. Required gradations generally are not standard production items. Some stone materials have a broad gradation band and most producers indicate little or no trouble producing these materials. However, sources that produce coarse aggregates for concrete may have trouble manufacturing or grading materials for the bedding. Contractors will be required to provide the selected sources adequate lead time to produce the various stone products, and the Contractor may propose more than one source for each of the materials.

6. **Material Weight.** The required minimum specific gravity for this project and Design Analysis level is 2.4 (or 150 pounds per cubic foot) for all materials.

7. **Material Quality.**

a. **General.** Quality requirements for each material type are discussed below. Riprap and larger spalls have been subjected to tests established by the Ohio River Division Laboratories, Cincinnati, OH. Tests No. P-11, "Riprap and Breakwater Stone Evaluation" includes a suite of tests to determine stone durability. The smaller size materials such as the smaller spalls and the bedding are included in ORDL Test Nos. C-21 and C-22, (Elementary Acceptance Tests for Fine Aggregates (C-21) and Coarse Aggregates (C-22) for Civil Works."

b. **Design Criteria.** Design criteria is a limiting factor on the number of available sources. Some producers will be eliminated from the list because their stone failed to meet the minimum specific gravity (SSD) of 2.4.

c. Type A Stone (Riprap, 1 to 700 Pounds). These stones will be a durable material, free from cracks, seams and overburden spoil. Only those sources from which the samples did not show any significant breakdown during the freeze-thaw and wet-dry tests are suitable. The freeze-thaw tests were performed for 35 cycles and the wet-dry tests for 80 cycles.

d. Type B Stone. (Riprap, 1 to 300 pounds). These stones will be a durable material, free from cracks, seams and overburden spoil. Only those sources from which the samples did not show any significant breakdown during the freeze-thaw and wet-dry tests are suitable. The freeze-thaw tests were performed for 35 cycles and the wet-dry tests for 80 cycles.

e. Type C Stone. (Spalls, 1/2 inch - 8 inches). These stones will be a reasonably durable, clean material free from cracks, seams, overburden spoil, and other deleterious materials. Only those sources from which the samples did not show any significant breakdown or deterioration during the freeze-thaw, wet-dry, and ORD lab test No. C-22 tests are suitable.

f. Type D Stone. (Bedding Material, No. 200 Sieve to 4 inches). This material will be a reasonably durable stone, clean and free from overburden spoil, shale, siltstone and other deleterious materials. Only those sources that did not show any significant deterioration in the ORD Lab Test Nos. C-21 or C-22 are suitable.

#### POSSIBLE SOURCES

8. The required stone materials to construct the facilities can be produced from the sources indicated on plates 1 through 7, "Possible Sources." These sources may be revised for the plans and specifications. However, all material from those sources may not be suitable. The right will be reserved in the specification to reject materials from certain localized areas, zones, strata, channels, or stockpiles when such materials are determined as unsuitable.

9. It is anticipated that selective quarrying will be required for some material types. Blasting techniques used for normal production will require adjustments or in some cases complete tailoring to produce riprap. The specifications shall state that the Contractor require the source to designate lifts, beds, and/or areas of the quarry for the production of riprap. Seasonal blasting and stockpiling of materials will be required prior to delivery at the project. Also, the specifications will require that shale and other undesirable materials will be excluded by adequate processing. All sources proposed by the Contractor will be subject to retesting prior to use in the project.

10. Twenty (20) sources are capable of producing the required materials. Transportation and logistics may be a problem for some of the smaller quarry operators as railheads and loading docks are some miles from the

quarry. Truckers often are reluctant to transport larger materials due to damage of truck beds.

11. Riprap.

a. Type A (1-700 pounds). Nine sources are listed. Three of these are within 32 miles of the project.

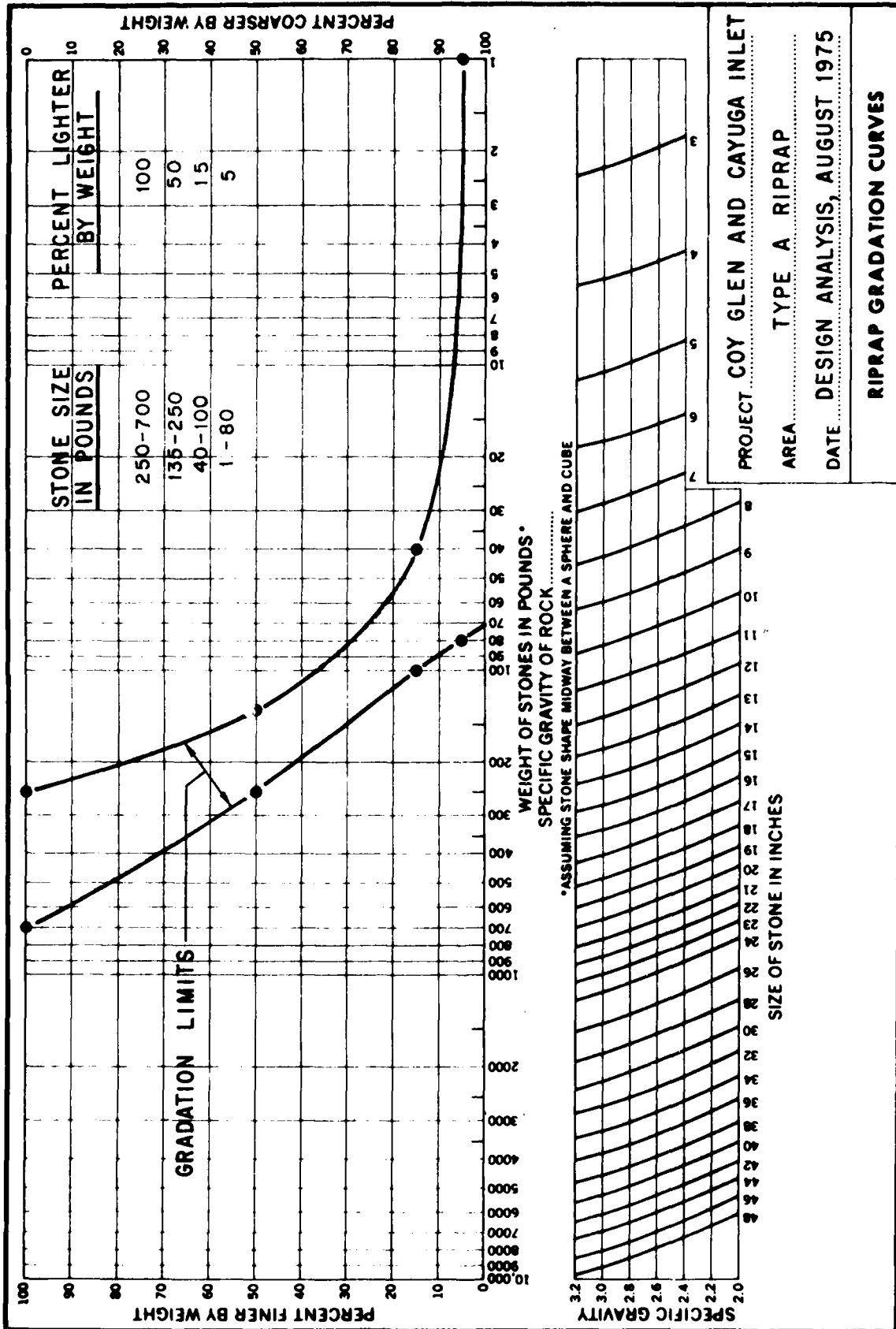
b. Type B (1-300 Pounds). Eleven sources are listed. Three of these are within 32 miles of the project.

12. Spalls. (Type C, 1/2 - 8 inches). Fifteen sources are listed. Three of these are within 32 miles of the project.

13. Bedding Material. (Type D, No. 200-4 inches sieve). Nineteen sources are listed. Three of these are within 32 miles of the project.

14. Riprap was used for both the Cayuga Inlet and Wellsville Rectification Projects. Cayuga Crushed Stone supplied stone to Cayuga Inlet in 1965, 1967, and 1968. Brown Quarry was opened in 1968 to supply additional stone. General Crushed Stone Inc., Honeoye, supplied riprap to the Wellsville Rectification Project in 1971. Only specific ledges in some quarries can produce the required size for riprap. For example, the basal 4 feet at Brown Quarry is too thin-bedded for use as a riprap material. Some quarries will require selective quarrying and productivity may be a problem.

15. Both spalls and bedding gradations are not standard production items and producers will be required to change screens or to blend available gradations.



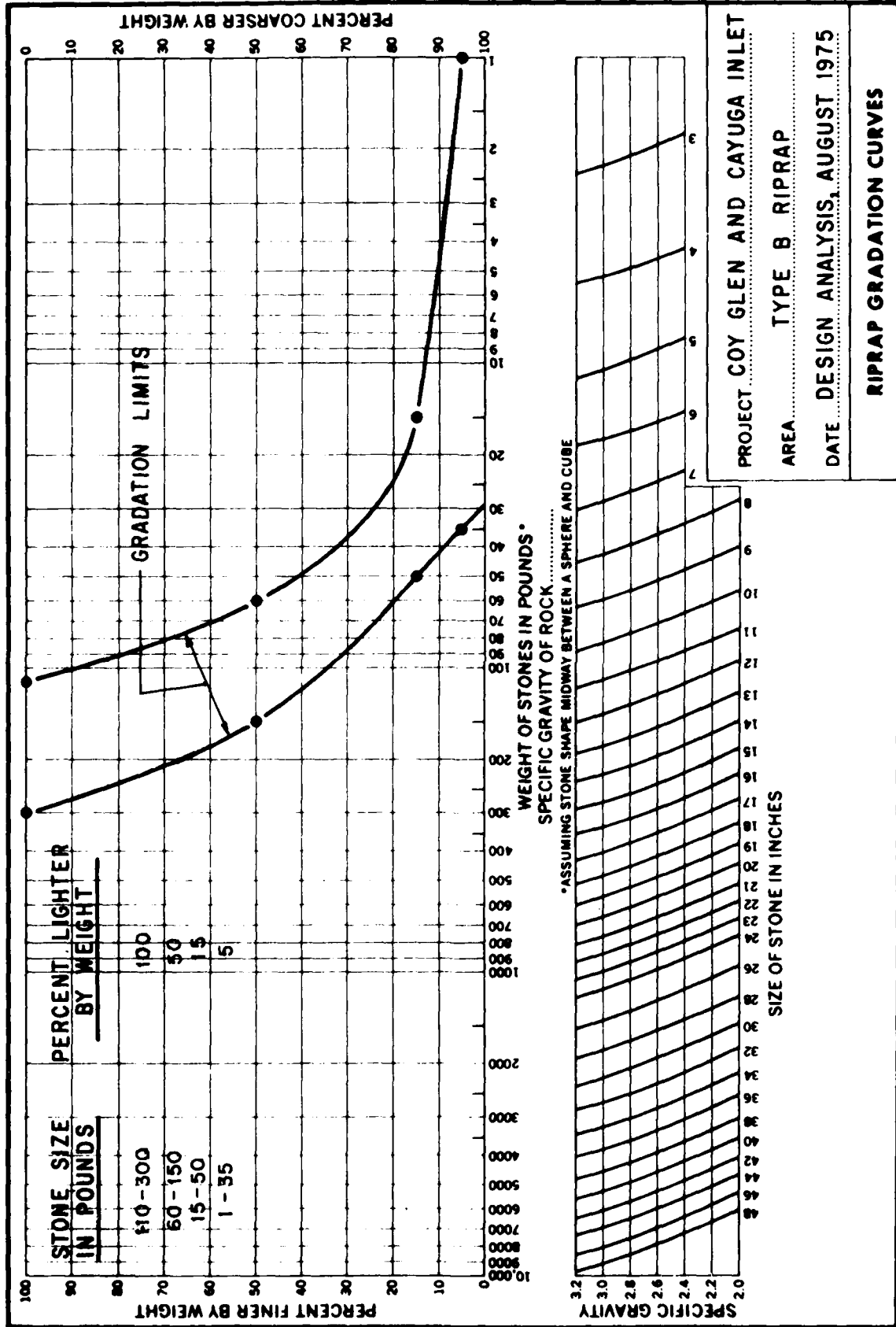
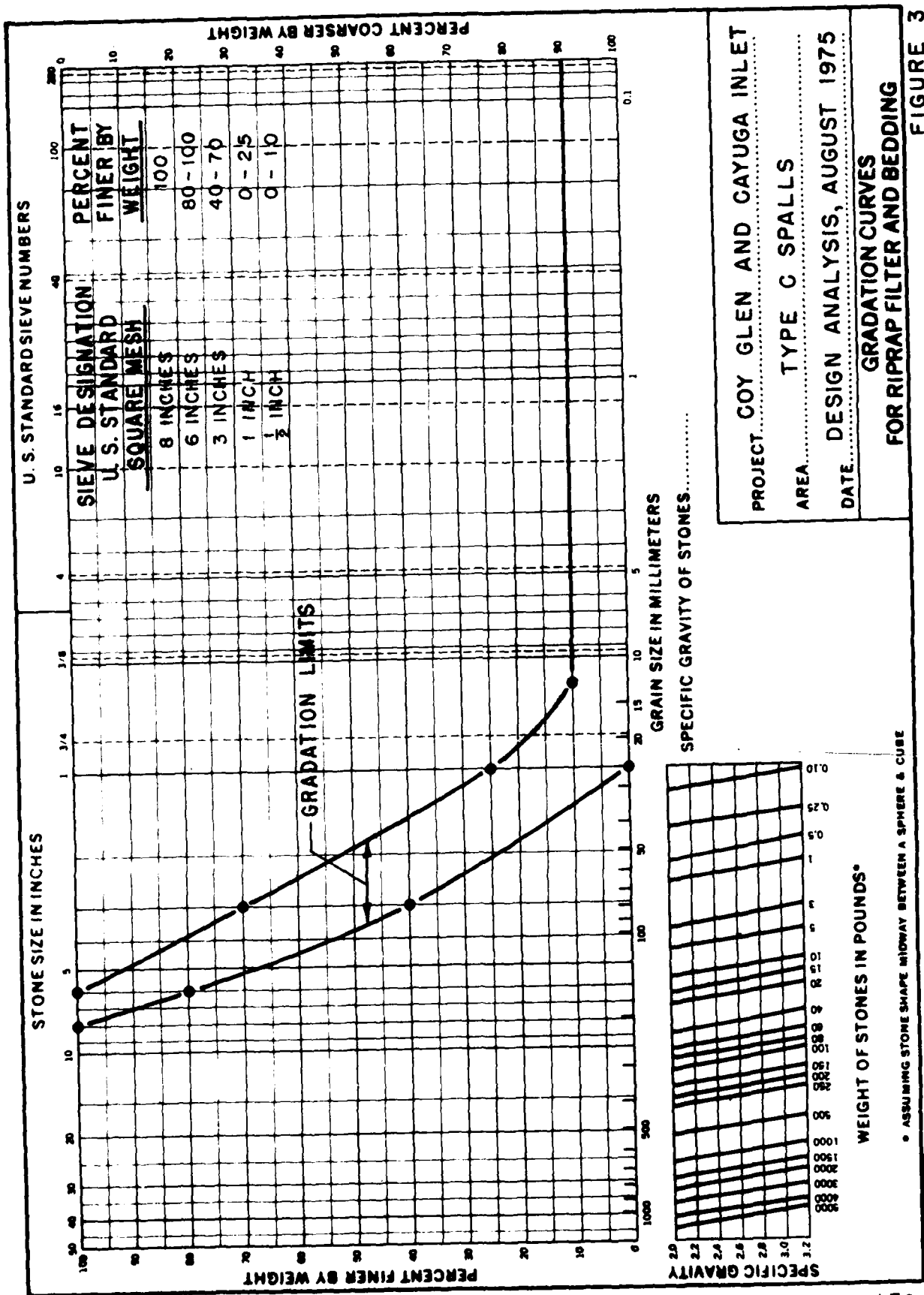
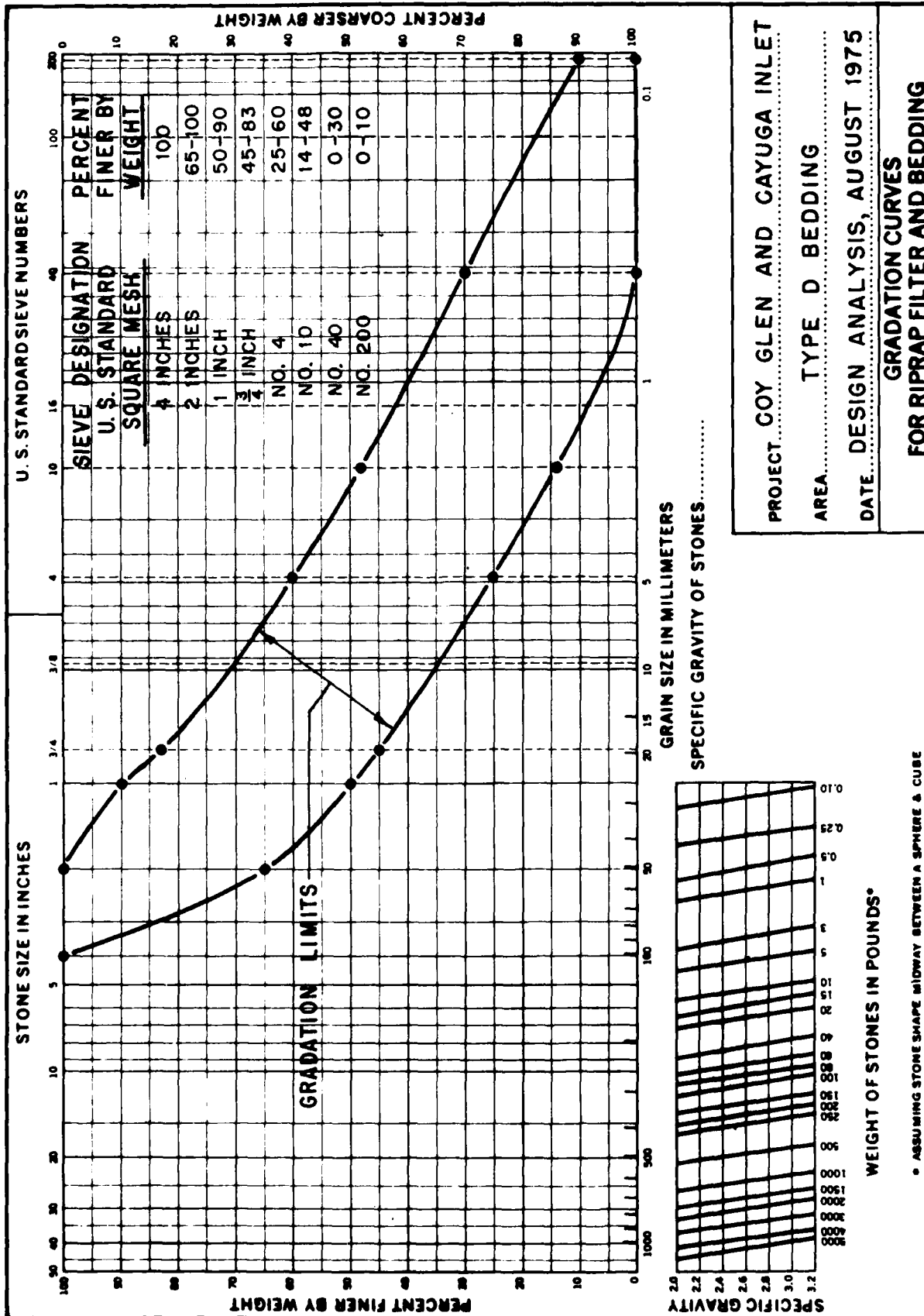


FIGURE 2







\* ASSUMING STONE SHAPE MIDWAY BETWEEN A SPHERE & CUBE

WEIGHT OF STONES IN POUNDS\*



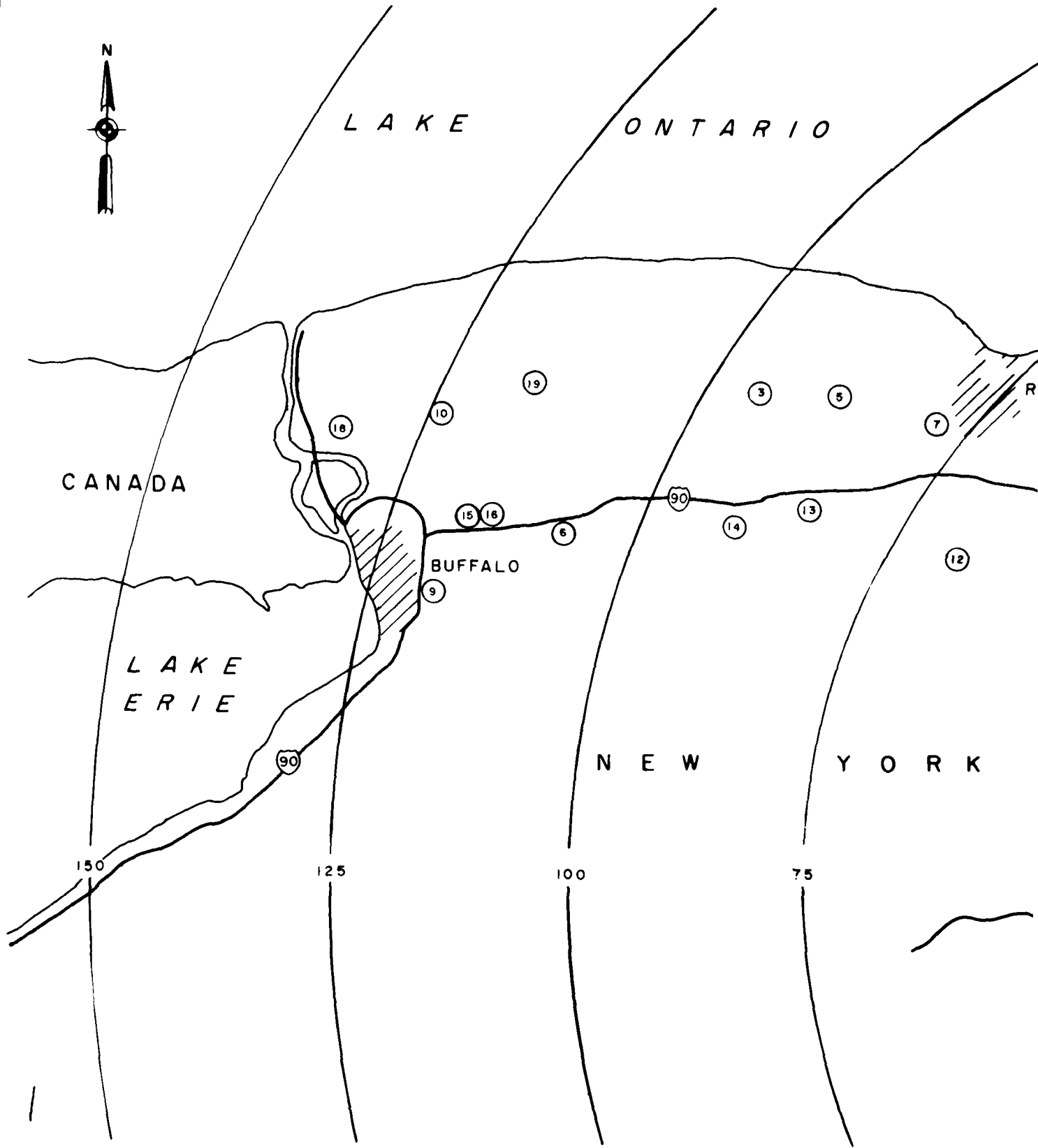
L A K E O N T A R I O

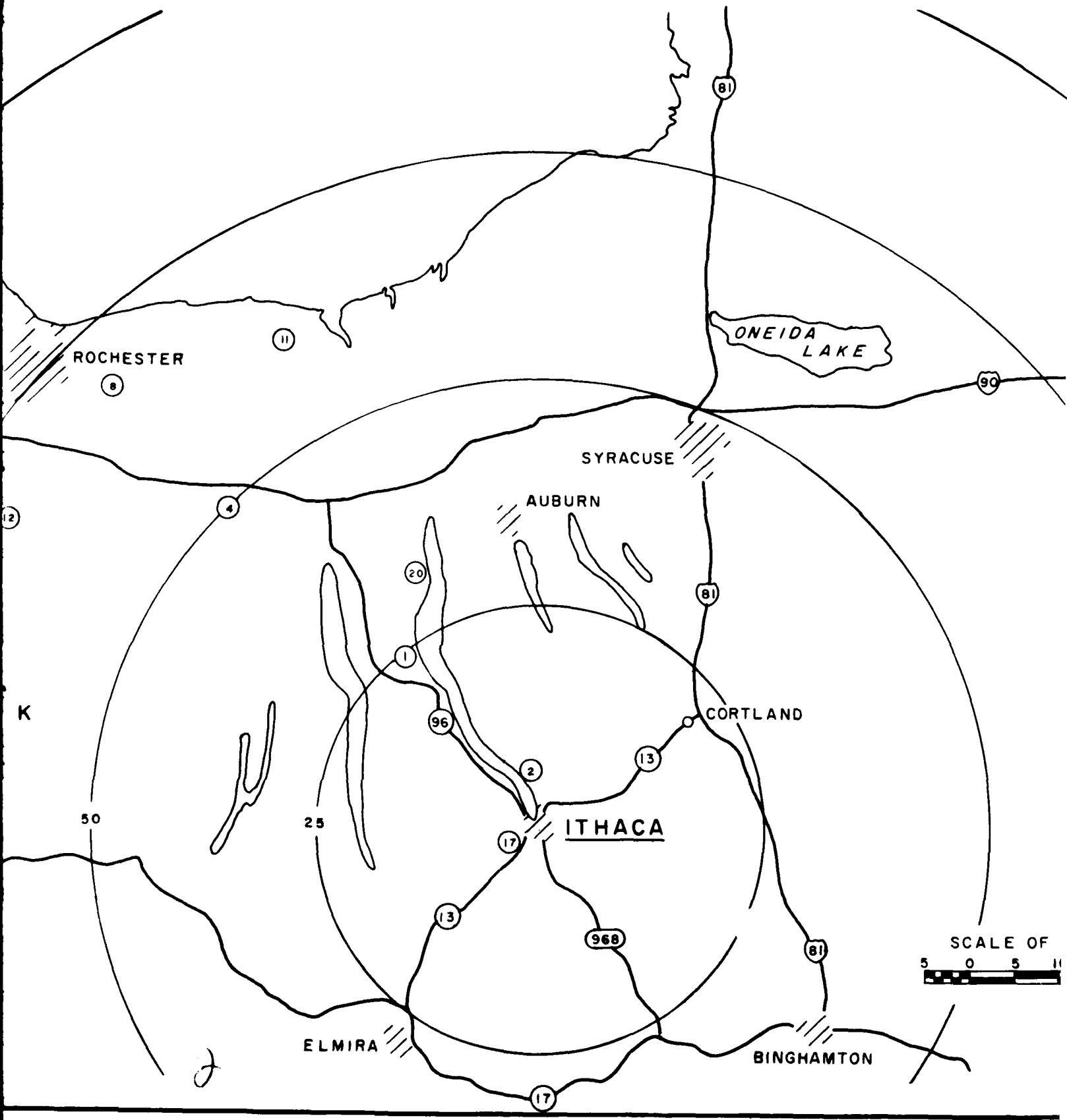
C A N A D A

L A K E  
E R I E

B U F F A L O

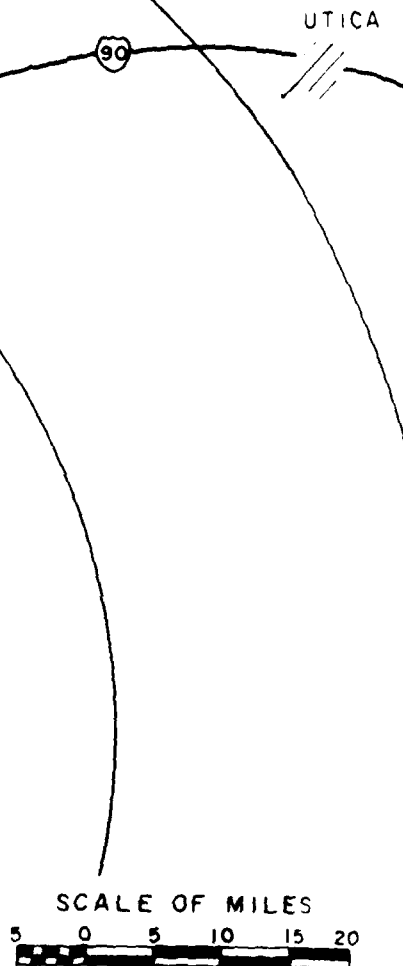
N E W Y O R K





NOTES:

1. NUMBER IN CIRCLE INDICATES QUARRY SITE.
2. FOR QUARRY NAMES AND PRODUCTS, SEE SUPPLEMENT SHEET.



COY GLEN AND CAYUGA INLET  
ITHACA, NEW YORK  
ENERGY DISSIPATOR FACILITIES  
AND RIPRAP REPAIR

**MATERIAL SURVEY**

U.S. ARMY ENGINEER DISTRICT, BUFFALO  
TO ACCOMPANY DESIGN ANALYSIS,  
AUGUST, 1975

MAP SUPPLEMENT SHEET  
SUMMARY OF POSSIBLE SOURCES FOR  
CONSTRUCTION MATERIALS

SITE NUMBER	SOURCE	QUARRY OR PIT LOCATION	RADIAL DISTANCE (IN MILES)					
				TYPE A RIPRAP	TYPE B RIPRAP	TYPE C SPALLS	TYPE D BEDDING	
1	BROWN QUARRY	OVID, N.Y.	23	X	X	X		
2.	CAYUGA CRUSHED STONE CO.	SOUTH LANSING, N.Y.	6	X	X	X	X	
3.	CLARENDON STONE PRODUCTS	CLARENDON, N.Y.	95				X	
4.	CONCRETE MATERIALS, INC.	MANCHESTER, N.Y.	49			X	X	
5.	CONCRETE MATERIALS, INC.	SWEDEN, N.Y.	88			X	X	
6.	COUNTY LINE STONE CO.	AKRON, N.Y.	107	X	X	X	X	
7.	DOLOMITE PRODUCTS, INC.	GATES CENTER, N.Y.	78		X	X	X	
8.	DOLOMITE PRODUCTS, INC.	PENFIELD, N.Y.	8		X	X	X	
9.	FEDERAL CRUSHED STONE CO.	CHEEKTOWAGA, N.Y.	119	X	X	X	X	
10.	FRONTIER STONE PRODUCTS	LOCKPORT, N.Y.	123	X	X	X	X	
11.	GENERAL CRUSHED STONE CO.	SODUS, N.Y.	51				X	
12.	GENERAL CRUSHED STONE CO.	HONEOYE, N.Y.	59		X	X	X	
13.	GENERAL CRUSHED STONE CO.	LERROY, N.Y.	84				X	
14.	GENESEE STONE PRODUCTS	STAFFORD, N.Y.	90		X	X	X	
15.	HOUDAILLE CONST. MTLs.	CLARENCE, N.Y.	116	X	X	X	X	
16.	LANCASTER STONE PRODUCTS	CLARENCE, N.Y.	115	X	X	X	X	
17.	LANDSTROM GRAVEL	ITHACA, N.Y.	3				X	
18.	NIAGARA STONE DIVISION	NIAGARA FALLS, N.Y.	132	X	X	X	X	
19.	ROYALTON STONE PRODUCTS	GASPORT, N.Y.	116			X	X	
20.	WARREN BROS.	CANOGA, N.Y.	32	X	X	X	X	



NOTES:

TYPE A RIPRAP - 1 LB. TO 700 POUNDS

TYPE B RIPRAP - 1 LB. TO 300 POUNDS

TYPE C SPALLS - 1/2 IN. TO 8 INCHES

TYPE D BEDDING - NO. 200 SIEVE TO 4 INCHES.

X - INDICATES THAT QUARRY OR PIT IS CAPABLE OF PRODUCING THAT MATERIAL.

COY GLEN AND CAYUGA INLET  
ITHACA, NEW YORK  
ENERGY DISSIPATOR FACILITIES  
AND RIPRAP REPAIR  
LOCATION MAP INDEX  
POSSIBLE MATERIAL SOURCES  
U.S. ARMY ENGINEER DISTRICT, BUFFALO  
TO ACCOMPANY DESIGN ANALYSIS, AUGUST 1975

3  
PLATE 2



POSSIBLE SOURCES FOR TYPES A,B,C AND D STONE

SOURCE	ROCK TYPE	PROPOSED USE	RADIAL DISTANCE	DATE
BROWN QUARRY QUARRY NEAR OVID, N.Y. OFFICE NEAR OVID, N.Y.	TULLY LIMESTONE	TYPES A,B AND C STONE	23 MI.	MARCH
CAYUGA CRUSHED STONE CO., INC. QUARRY AT SOUTH LANSING, N.Y. OFFICE AT SOUTH LANSING, N.Y.	TULLY LIMESTONE	TYPES A,B,C AND D STONE	6 MI.	MARCH
				SEPTEMBER
CLARENDON STONE PRODUCTS QUARRY AT CLARENDON, N.Y. OFFICE AT CLARENDON, N.Y.	LOCKPORT DOLOMITE	TYPE D STONE	95 MI.	MAY 19
CONCRETE MATERIALS, INC. QUARRY AT SWEDEN, N.Y. OFFICE AT SWEDEN, N.Y.	LOCKPORT DOLOMITE	TYPES C AND D STONE	88 MI.	JANUARY
1				

DIAL RANCE	LABORATORY TEST RECORD			DATE USED
	DATE TESTED	LABORATORY	PROJECT FOR WHICH TESTED	
MI.	MARCH 1969	ORD LAB LAB # 103/69.611C	CAYUGA INLET ITHACA, N.Y.	1969
MI.	MARCH 1967	ORD LAB LAB # 101/67.358C	CAYUGA INLET FLOOD PROTECTION PROJECT, ITHACA, N.Y.	1968
	SEPTEMBER 1965	ORD LAB LAB # 103/66.600C	CAYUGA INLET, STAGES I AND II	1965, 1967 AND 1968
MI.	MAY 1972	ORD LAB LAB # 103/72.610C	OAK ORCHARD HARBOR, N.Y.	UNKNOWN
MI.	JANUARY 1971	ORD LAB LAB # 101/71.362C	ROCHESTER HARBOR, N.Y. EAST PIER REPAIRS	1971
	2			

[illegible]

		REMARKS
ON		
	TESTING REQUIRED. SPECIFIC GRAVITY OF 2.72 BASAL 4 FEET NOT ACCEPTABLE PROCESSING EQUIPMENT MAYBE LACKING.	
	SPECIFIC GRAVITY IS 2.76.	
	UNIT WEIGHT AVERAGES 171.2 P.C.F. RAIL FACILITIES AVAILABLE. TESTING REQUIRED.	
	SPECIFIC GRAVITY IS 2.76. ONLY TRUCKING FACILITIES AVAILABLE.	
	SPECIFIC GRAVITY IS 2.75.	
		COY GLEN AND CAYUGA INLET ITHACA, NEW YORK ENERGY DISSIPATOR FACILITIES AND RIPRAP REPAIR  MATERIAL SURVEY  U. S. ARMY ENGINEER DISTRICT, BUFFALO TO ACCOMPANY DESIGN ANALYSIS, AUGUST 1975

POSSIBLE SOURCES FOR TYPES A,B,C AND D STONE					
SOURCE	ROCK TYPE	PROPOSED USE	RADIAL DISTANCE		
CONCRETE MATERIALS INC. QUARRY AT MANCHESTER, N.Y. OFFICE AT BROCKPORT, N.Y.	ONONDAGA FORMATION (LIMESTONE)	TYPES C AND D STONE	49 MI.		AUGU
COUNTY LINE STONE CO. QUARRY AT AKRON, N.Y. OFFICE AT AKRON, N.Y.	ONONDAGA FORMATION (LIMESTONE)	TYPES A,B,C AND D STONE	107 MI.		MAY
					FEBR
					SEPT
DOLOMITE PRODUCTS QUARRY AT GATES CENTER, N.Y. OFFICE AT ROCHESTER, N.Y.	LOCKPORT FORMATION (DOLOMITE)	TYPES B,C AND D STONE	78 MI.		MAY
DOLOMITE PRODUCTS QUARRY AT PENFIELD, N.Y. OFFICE AT PENFIELD, N.Y.	LOCKPORT FORMATION (DOLOMITE)	TYPES B,C, AND D STONE	68 MI.		UNKN
					JUNE

**JUNE**

RADIAL DISTANCE	LABORATORY TEST RECORD			DATE USED
	DATE TESTED	LABORATORY	PROJECT FOR WHICH TESTED	
49 MI.	AUGUST 1973	ORD LAB LAB # 103/73.630C	CONFINED DREDGE SPOIL DISPOSAL PROGRAM (RIPRAP)	UNKNOWN
107 MI.	MAY 1967	ORD LAB LAB # 103/67.605C	WARSAW N.Y. FLOOD CONTROL PROJECT (RIPRAP)	1967
	FEBRUARY 1971	ORD LAB LAB # 103/71.612C	BUFFALO DIKED DISPOSAL AREA #2 (RIPRAP)	1971
	SEPTEMBER 1974	ORD LAB	CONFINED DREDGE SPOIL DISPOSAL AREAS NOS. 1 AND 2, BUFFALO HARBOR NEW YORK (REPAIRS)	
78 MI.	MAY 1972	ORD LAB LAB # 103/72.610C	OAK ORCHARD HARBOR, N.Y. (CORE STONE, COVER STONE AND CONCRETE AGGREGATE)	UNKNOWN
68 MI.	UNKNOWN	UNKNOWN	BUFFALO DIKED DISPOSAL AREA #2 (RIPRAP)	UNKNOWN
	JUNE 1973	ORD LAB LAB # 103/73.603C	CONFINED DREDGE SPOIL DISPOSAL PROGRAM	UNKNOWN

[illegible]

[illegible]



POSSIBLE SOURCES FOR TYPES A,B,C AND D STONE

SOURCE	ROCK TYPE	PROPOSED USE	RADIAL DISTANCE	
FEDERAL CRUSHED STONE DIV. OF BUFFALO SLAG CO. INC., QUARRY AT CHEEKTOWAGA N.Y., OFFICE AT BUFFALO N.Y.	ONONDAGA FORMATION (LIMESTONE)	TYPES A,B,C AND D STONE	119 MI.	NOV
				FEB
				MAR
				APR
FRONTIER STONE PRODUCTS, INC. QUARRY AT LOCKPORT, N.Y. OFFICE AT LOCKPORT, N.Y.	LOCKPORT FORMATION (DOLOMITE)	TYPES A,B,C AND D STONE	123 MI.	FEB
				AUG
GENERAL CRUSHED STONE INC. QUARRY AT SODUS, N.Y. OFFICE AT EASTON, PA.	LOCKPORT FORMATION (DOLOMITE)	TYPE D STONE	61 MI.	MAY
				FEB
				JUN
				JAN
1				

DIAL ANCE	LABORATORY TEST RECORD			DATE USED
	DATE TESTED	LABORATORY	PROJECT FOR WHICH TESTED	
9 MI.	NOVEMBER 1965	ORD LAB LAB # 103/66.605C	LOCAL FLOOD PROTECTION PROJECT. SMOKES CREEK, STAGE 11 (RIPRAP)	UNKNOWN
	FEBRUARY 1971	ORD LAB LAB # 103/71.612C	BUFFALO DIKED DISPOSAL AREA #2 (RIPRAP)	UNKNOWN
	MARCH 1972	ORD LAB LAB # 103/72.606C	CONFINED DIKE DISPOSAL PROGRAM (CONCRETE AGGREGATE)	UNKNOWN
	APRIL 1973	ORD LAB LAB # 103/73.337C	BLACK ROCK LOCK REHABILITATION	MAY 1973
MI.	FEBRUARY 1971	ORD LAB LAB # 103/71.612C	BUFFALO DIKED DISPOSAL AREA #2 (RIPRAP)	UNKNOWN
	AUGUST 1974	UNKNOWN	CONFINED DIKE DISPOSAL PROGRAM, BUFFALO HARBOR, N.Y., SITE 4 (ARMOR STONE)	UNKNOWN
MI.	MAY 1971	ORD LAB LAB # 101/71.358C	LITTLE SODUS BAY, N.Y. PIER REPAIR (CONCRETE AGGREGATE)	UNKNOWN
	FEBRUARY 1972	ORD LAB LAB # 103/72.607C	LITTLE SODUS BAY, N.Y. PIER REPAIR (CONCRETE AGGREGATE)	UNKNOWN
	JUNE 1973	ORD LAB LAB # 103/73.630C	CONFINED DIKE DREDGE DISPOSAL PROGRAM (RIPRAP)	UNKNOWN
	JANUARY 1974	ORD LAB LAB # 103/74.613C	LITTLE SODUS BAY, N.Y. PIER REPAIR (CONCRETE AGGREGATE)	UNKNOWN
	2			

# SERVICE RECORD

USED	PROJECT	EVALUATION	
	UNKNOWN	UNKNOWN	UNIT WEIGHT AVERAGES 168 P.C.F
	UNKNOWN	UNKNOWN	ONLY THE FIRST LIFT. WEST QUAR P.C.F. TO 169 P.C.F. RAIL FAC
	UNKNOWN	UNKNOWN	SPECIFIC GRAVITY VARIES FROM 2
	BLACK ROCK LOCK REHABILITATION	SOME POPOUTS AND SPALLING (1975)	TYPE II, LOW ALKALI CEMENT REC
	UNKNOWN	UNKNOWN	THE DECEW MEMBER NOT ACCEPTABI FROM 162 P.C.F. RAIL FACILIT
	UNKNOWN	UNKNOWN	ONLY THE GASPORT MEMBER ACCEP ON NYS BARGE CANAL TO BE AVAI DECEW MEMBER CURRENTLY BEING WILL REQUIRE TESTING.
	UNKNOWN	UNKNOWN	ALL CRUSHED MATERIALS WILL RE
	UNKNOWN	UNKNOWN	
	UNKNOWN	UNKNOWN	
	UNKNOWN	UNKNOWN	
	UNKNOWN	UNKNOWN	
	UNKNOWN	UNKNOWN	
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	REMARKS
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	UNIT WEIGHT AVERAGES 168 P.C.F.
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	ONLY THE FIRST LIFT, WEST QUARRY TESTED. UNIT WEIGHT VARIES FROM 166 P.C.F. TO 169 P.C.F. RAIL FACILITIES NOT AVAILABLE.
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	SPECIFIC GRAVITY VARIES FROM 2.68 TO 2.70. LOW ALKALI CEMENT REQUIRED.
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LING	TYPE II, LOW ALKALI CEMENT REQUIRED.
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	THE DECEW MEMBER NOT ACCEPTABLE FOR THIS PROJECT. UNIT WEIGHTS VARY FROM 162 P.C.F. RAIL FACILITIES NOT AVAILABLE.
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	ONLY THE GASPORT MEMBER ACCEPTABLE FOR ARMOR STONE. LOADING FACILITIES ON NYS BARGE CANAL TO BE AVAILABLE. SELECTIVE QUARRYING REQUIRED. DECEW MEMBER CURRENTLY BEING RETESTED (JULY 1975). CRUSHED MATERIALS WILL REQUIRE TESTING.
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	ALL CRUSHED MATERIALS WILL REQUIRE RETESTING.
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		COY GLEN AND CAYUGA INLET ITHACA, NEW YORK ENERGY DISSIPATOR FACILITIES AND RIPRAP REPAIR MATERIAL SURVEY

		U. S. ARMY ENGINEER DISTRICT, BUFFALO TO ACCOMPANY DESIGN ANALYSIS, AUGUST 1975
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POSSIBLE SOURCES FOR TYPES A,B,C AND D STONE

SOURCE	ROCK TYPE	PROPOSED USE	RADIAL DISTANCE	
GENERAL CRUSHED STONE CO. QUARRY AT HONEOYE FALLS. N.Y. OFFICE AT EASTON. PA.	ONONDAGA FORMATION (LIMESTONE)	TYPES A,B,C AND D STONE	68 MI.	DEC
GENERAL CRUSHED STONE CO. QUARRY AT LEROY. N.Y. OFFICE AT EASTON. PA.	ONONDAGA FORMATION (LIMESTONE)	TYPE D STONE	84 MI.	DEC
GENESEE STONE PRODUCTS CORP. QUARRY AT STAFFORD. N.Y. OFFICE AT BATAVIA. N.Y.	ONONDAGA FORMATION (LIMESTONE)	TYPES C AND D STONE	90 MI.	DEC
				JAN
HOUDAILLE CONSTRUCTION MATERIALS. INC. QUARRY AT CLARENCE. N.Y. OFFICE AT CLARENCE. N.Y.	ONONDAGA FORMATION (LIMESTONE)	TYPES A,B,C AND D STONE	116 MI.	JUL
				SEP
				FEB
				APR
/				

# LABORATORY TEST RECORD

DATE TESTED	LABORATORY	PROJECT FOR WHICH TESTED	DATE USED	
DECEMBER 1971	ORD LAB LAB # 103/72.602C	WELLSVILLE RECTIFICATION PROJECT, WELLSVILLE N.Y. (RIPRAP)	1971	WELLSVILLE PROJECT (R
DECEMBER 1971	ORD LAB LAB # 103/72 602C	WELLSVILLE RECTIFICATION PROJECT, WELLSVILLE, N.Y. (RIPRAP)	UNKNOWN	UNKNOWN
DECEMBER 1971	ORD LAB LAB # 103/72.602C	WELLSVILLE RECTIFICATION PROJECT, WELLSVILLE, N.Y. (RIPRAP)	UNKNOWN	UNKNOWN
JANUARY 1974	ORD LAB LAB # 103/74.610C	WELLSVILLE RECTIFICATION PROJECT, WELLSVILLE, N.Y. (RIPRAP)	UNKNOWN	UNKNOWN
JULY 1959	ORD LAB LAB # 412/59Z	NORTH ENTRANCE, BUFFALO HARBOR, N.Y. (CORE STONE)	UNKNOWN	UNKNOWN
SEPTEMBER 1965	ORD LAB LAB # 103/66.602C	LOCAL FLOOD PROTECTION PROJECT, SMOKES CREEK, STAGE II, (RIPRAP)	UNKNOWN	UNKNOWN
FEBRUARY 1971	ORD LAB LAB # 103/71.612C	BUFFALO DIKED DISPOSAL AREA #2 (RIPRAP AND SPALLS)	1971	BUFFALO D (RIPRAP A
APRIL 1972	ORD LAB LAB # 103/72.606C	CONFINED DREDGE SPOIL DISPOSAL PROGRAM (CONCRETE AGGREGATE)	UNKNOWN	UNKNOWN
2				

SERVICE RECORD			REMARKS
TESTED	PROJECT	EVALUATION	
	WELLSVILLE EMERGENCY FLOOD CONTROL PROJECT (RIPRAP)	SATISFACTORY	QUARRY NOT RESPONSIBLE FOR GRADATION P.C.F. TO 168 P.C.F. RAIL FACILITIES CRUSHED MATERIAL WILL REQUIRE TESTING
	UNKNOWN	UNKNOWN	UNIT WEIGHT AVERAGES 167 P.C.F. QU UNIFORM SIZE RIPRAP. RAIL FACILITIES CRUSHED MATERIALS WILL REQUIRE TESTING
	UNKNOWN	UNKNOWN	ONLY THE FIRST AND SECOND LIFT ACCEPTED 168 P.C.F. RAIL FACILITIES NOT AVAILABLE
	UNKNOWN	UNKNOWN	THE THIRD LIFT IS NOT ACCEPTABLE.
	UNKNOWN	UNKNOWN	CRUSHED MATERIALS WILL REQUIRE TESTING
	UNKNOWN	TOO THIN BEDDED FOR USE ON PROJECT TESTED FOR	
	BUFFALO DIKE DISPOSAL AREA #2 (RIPRAP AND SPALLS)	TOO EARLY TO EVALUATE	ONLY THE SECOND LIFT TESTED AND USED P.C.F. TO 171 P.C.F. RAIL FACILITIES
	UNKNOWN	UNKNOWN	NOT RECOMMENDED FOR USE AS CONCRETE REQUIRED.
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REMARKS	
	QUARRY NOT RESPONSIBLE FOR GRADATION. UNIT WEIGHT VARIES FROM 166 P.C.F. TO 168 P.C.F. RAIL FACILITIES NOT AVAILABLE. CRUSHED MATERIAL WILL REQUIRE TESTING
	UNIT WEIGHT AVERAGES 167 P.C.F. QUARRY WILL NOT PROCESS A GRADED OR UNIFORM SIZE RIPRAP. RAIL FACILITIES AVAILABLE. CRUSHED MATERIALS WILL REQUIRE TESTING.
	ONLY THE FIRST AND SECOND LIFT ACCEPTABLE. UNIT WEIGHT AVERAGES 168 P.C.F. RAIL FACILITIES NOT AVAILABLE.
	THE THIRD LIFT IS NOT ACCEPTABLE.
	CRUSHED MATERIALS WILL REQUIRE TESTING.
	ONLY THE SECOND LIFT TESTED AND USED. UNIT WEIGHT VARIES FROM 165 P.C.F. TO 171 P.C.F. RAIL FACILITIES AVAILABLE.
	NOT RECOMMENDED FOR USE AS CONCRETE AGGREGATE. LOW ALKALI CEMENT REQUIRED.
	COY GLEN AND CAYUGA INLET ITHACA, NEW YORK ENERGY DISSIPATOR FACILITIES AND RIPRAP REPAIR MATERIAL SURVEY U. S. ARMY ENGINEER DISTRICT, BUFFALO TO ACCOMPANY DESIGN ANALYSIS, AUGUST 1975



POSSIBLE SOURCES FOR TYPES A,B,C AND D STONE

SOURCE	ROCK TYPE	PROPOSED USE	RADIAL DISTANCE	
LANCASTER STONE PRODUCTS CORP. QUARRY AT CLARENCE, N.Y. OFFICE AT WILLIAMSVILLE, N.Y.	ONONDAGA FORMATION (LIMESTONE)	TYPES A,B,C AND D STONE	115 MI.	OCTO
LANDSTROM GRAVEL PIT PIT AT ITHACA, N.Y. OFFICE AT ITHACA, N.Y.	GLACIAL DEPOSIT	TYPE D STONE	3 MI.	UNKI
NIAGARA STONE DIV. OF GREAT LAKES COLOR PRINTING CORP., QUARRY AT NIAGARA FALLS, N.Y. (PLETCHERS CORNERS) OFFICE AT NIAGARA FALLS, N.Y.	LOCKPORT FORMATION (DOLOMITE)	TYPES A,B,C AND D STONE	132 MI.	FEBF
ROYALTON STONE PRODUCTS, INC. QUARRY AT GASPORT, N.Y. OFFICE AT GASPORT, N.Y.	LOCKPORT FORMATION (DOLOMITE)	TYPES C AND D STONE	116 MI.	FEBF
WARREN BROS. QUARRY AT CANOGA, N.Y. OFFICE AT GENEVA, N.Y.	ONONDAGA FORMATION (LIMESTONE)	TYPES A,B,C AND D STONE	32 MI.	OCTI
1				

# LABORATORY TEST RECORD

DATE TESTED	LABORATORY	PROJECT FOR WHICH TESTED	DATE USED	
OCTOBER 1967	ORD LAB LAB * 103/68.605C	BUFFALO DIKED DISPOSAL AREA #1 (RIPRAP)	UNKNOWN	UNKN
UNKNOWN	UNKNOWN	UNKNOWN	UNKNOWN	UNKN
FEBRUARY 1971	ORD LAB LAB * 103/71.612C	BUFFALO DIKED DISPOSAL AREA #2 (RIPRAP)	UNKNOWN	UNKN
FEBRUARY 1971	ORD LAB LAB * 103/71.612C	BUFFALO DIKED DISPOSAL AREA #2 (RIPRAP)	UNKNOWN	UNKN
OCTOBER 1968	ORD LAB LAB * 103/74.601C	GREAT SODUS HARBOR, N.Y. EMERGENCY WEST PIER REPAIR (BREAKWATER STONE)	UNKNOWN	UNKN
2				

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HOWARD NEEDLES TAMMEN AND BERGENOFF NEW YORK  
ENERGY DISSIPATOR FACILITIES AND RIPRAP REPAIR, COY GLEN AND CA--ETC  
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F/G 13/10  
COY GLEN AND CA--ETC  
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# SERVICE RECORD

PROJECT

EVALUATION

REMARKS

UNKNOWN

UNKNOWN

ONLY THE LOWER LIFT TESTED (1967). UNIT W  
TO 169 P.C.F. RAIL FACILITIES NOT AVAILAB.  
CRUSHED MATERIALS WILL REQUIRE TESTING.

UNKNOWN

UNKNOWN

TESTING REQUIRED. AN ACCEPTABLE SOURCE FO

UNKNOWN

UNKNOWN

BOTH LIFTS CONSISTING OF OAK ORCHARD. ERAM  
MEMBERS ACCEPTABLE. UNIT WEIGHT VARIES FR  
RAIL FACILITIES AVAILABLE. MANAGEMENT MAY  
SIZE MATERIAL, CRUSHED MATERIALS REQUIRE TE

UNKNOWN

UNKNOWN

ONLY MATERIALS FROM EAST END OF QUARRY TES  
FROM 163 P.C.F. TO 165 P.C.F. RAIL FACILI  
CRUSHED MATERIALS REQUIRE TESTING

UNKNOWN

UNKNOWN

UNIT WEIGHT VARIES FROM 166 P.C.F. TO 169  
CRUSHED MATERIALS REQUIRE TESTING.

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	REMARKS
	ONLY THE LOWER LIFT TESTED (1967). UNIT WEIGHT VARIES FROM 166 P.C.F. TO 169 P.C.F. RAIL FACILITIES NOT AVAILABLE. CRUSHED MATERIALS WILL REQUIRE TESTING.
	TESTING REQUIRED. AN ACCEPTABLE SOURCE FOR CAYUGA INLET, STAGE III.
	BOTH LIFTS CONSISTING OF OAK ORCHARD, ERAMOSA AND UPPER GOAT ISLAND MEMBERS ACCEPTABLE. UNIT WEIGHT VARIES FROM 166 P.C.F. TO 174 P.C.F. RAIL FACILITIES AVAILABLE. MANAGEMENT MAY BE RELUCTANT TO PRODUCE LARGE SIZE MATERIAL, CRUSHED MATERIALS REQUIRE TESTING.
	ONLY MATERIALS FROM EAST END OF QUARRY TESTED. UNIT WEIGHT VARIES FROM 163 P.C.F. TO 165 P.C.F. RAIL FACILITIES AVAILABLE. CRUSHED MATERIALS REQUIRE TESTING
	UNIT WEIGHT VARIES FROM 166 P.C.F. TO 169 P.C.F. CRUSHED MATERIALS REQUIRE TESTING.

COY GLEN AND CAYUGA INLET  
ITHACA, NEW YORK  
ENERGY DISSIPATOR FACILITIES  
AND RIPRAP REPAIR

MATERIAL SURVEY

U. S. ARMY ENGINEER DISTRICT, BUFFALO  
TO ACCOMPANY DESIGN ANALYSIS, AUGUST 1975

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